A STUDY OF THE WATER POWER DEVELOPMENT IN CONNECTION WITH THE RESERVOIRS FOR FLOOD CONTROL, YUNG.TING HO, CHINA

by

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Signature of Author...
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Cambridge, Massachusetts

Sept. 23, 1935

Prof. G. W. Swett,
Secretary of the Faculty,
Massachusetts Institute of Technology,
Cambridge, Massachusetts.

Dear Sir:

I submit herewith my thesis entitled "A Study of the Water Power Development in Connection with the Reservoirs for Flood Control, Yung Ting Ho, China" in partial fulfillment of the requirement for the degree of Master of Science in Civil Engineering.

Respectfully yours

Chang-Ling Chang

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CONTENTS

Chapter ... Page

I. The Combined Use of Reservoir and its Application to Yung Ting Project.

Synopsis 1- 4

A description of Yung Ting Valley and the project, 4-11

Value of Yung Ting water power 11-17

The combined use of Kuan Ting reservoir 17-25

II. The Economy of Yung Ting Power Development---Preliminary Investigation

The cost of flood disposal 26-31

Cost of dam 31-32

The limiting height of dam 32-34

The waterway 34-35

III. Cost and Value of Yung Ting Power Development

The power plants 36-38

Cost and value of plant 2 38-41

Cost and value of plant 3 41-43

Conclusion and suggestions 43-46

Computation and Design 47-67

Bibliography 68

Figures 1-19

Appendix. Hydrographs, Water Level and Rating Curves, Kuan-Ting Station, Yung Ting Ho, 1925-1934

I. The Combined Use of Reservoir and Its Application to the Yung Ting Project

A. Synopsis.

There are always some conflictions between flood control and power reservoirs, a reservoir for flood prevention should be kept as nearly empty as possible to absorb all the flood flow, a reservoir for power use should be kept full to create head or to provide water during dry seasons. In some cases, however, their combined use may be proved to be more economical; the highest flood of some streams may always occur in spring due to heavy rainfall combined with melting snow, others may always occur in summer caused solely by great storms; in these circumstances the reservoir may be kept empty for certain period and used for power purpose the rest time of the year. The reduction of flood peak and consequently the equalization of flow by flood-control reservoirs may increase the power output of the downstream plants and also lessen the quantity of flood to be handled by those plants. Large reservoirs for power use may absorb or retard some part of the flood and greatly reduce its peak. The cost of reservoir per unit volume of storage usually decreases with increasing capacity; the reservoir may be built such that a portion of which is used for flood control while the remaining is used for power purpose; the cost of combined reservoirs is usually cheaper than two separate ones.

It is the aim of some designers to have their flood-control project in connection with other purposes such as power, irrigation etc. to share the construction and maintenance expenses. On the Report of Advisory Committee of Engineers on Flood Control Vt. it is stated that upon all the rivers in Vt. storage capacity has been found sufficient practically to eliminate the flood hazard; the estimated total cost of reservoirs is 40,000,000 dollars and the development of 280,000 hp. at or near the reservoir sites will cost $34,000,000 more; the average cost of power is estimated to be .8¢ per kw-hr. including all fixed and operating charges for reservoirs and power developments. (1)

The report of New York Water Power Commission ~~on~~ on water power and storage possibilities of the Hudson River states that the flood flow of 89,000 c.f.s. at Spier Falls March, 1913 could be reduced by about 35,000 c.f.s., if the Sacandaga power reservoir had been in operation; and the max. stage at Albany might have been reduced by 5 ft., if the proposed system of reservoirs had been in operation, and hence the prevention of flooding of the Albany Filtration Plant which caused the serious typhoid epidemic immediately after that flood. (2)

The Tennessee Valley Project provides 5 ft. surcharge on the reservoirs on tributaries and 10 ft. on those in the main stream for flood protection; by this means the flood can be prevented at a very low cost.(3)

Out of the 30 mill. acre-ft. storage capacity of Boulder Reservoir Colorado River, 9.5 mill. acre-ft. is used for flood control, this can reduce the highest peak flood of 250,000 c.f.s. to 50,000 c.f.s. Although 25 mill. dollars is attributed to this work, it is estimated that in 50 years the return from the power plant alone amounts more than 165 mill. dollars, the cost of the whole project for irrigation, flood control and power. (4)

As to the effect of power reservoirs on flood the report of the Committee on Floods of New England, Boston Society C. E. says that except for a period in the late Spring and early summer the water level in power reservoirs is normally below spillway level,and there will be more or less capacity to absorb floods; that storage above spillway crest is also available to cut down flood peaks; and that by proper operation of reservoirs such as to open the flood gates a short time before the arrival of flood, the flood flow may be evenly distributed and its peak greatly reduced. (5)

For the control of Yung Ting Ho, a river which offen causes disastrous floods, the North China River Commission has proposed the construction of two reservoirs (Kuan Ting and Tai Tsu Mu), it is the purpose of this paper to study the possibilities of water power development in connection with these reservoirs. It is found that:

1. The Kuan Ting reservoir can be used also for power, the storage for this purpose will be 5.26 bill. cu. ft. from Sept. to June and 0.795 bill. cu. ft. in July and August. The Kuan Ting Dam will be raised 4.1 ft. and the cost of storage including control works will be $180,000.

2. The power plants will be located on the portion of Yung Ting Ho between Kuan Ting and San Chia Tien, out of the total drop of 1,100 ft. in about 70 miles, around 600 ft. are available for power and there will be altogether 72,000 hp. which may be developed in about 6 plants.

3. The investment per hp. will be from $100 to 150, depending on the site and size of the plant; the cost per kw-hr. will be from .38 to .54¢, while the value of power is estimated to be .55¢ per kw-hr.

4. Based on market conditions it is considered that a 4000-6,000 kw. plant may be built presently or in the near future, the cost of this plant will be around $110-130 per hp. including cost of storage and the cost of power will be about .4¢ per kw-hr.

B. A description of Yung Ting Valley and the flood control project

<u>The Yung Ting Valley</u> Fig. 1 is a sketch of Yung Ting Valley, the upper Yung Ting is a very dry, cold and moun-

tainous region, the temperature may reach -20°C in winter; the annual rainfall is usually but a little more than 12 inches which makes cultivation very difficult; the land is sandy and highly erodible. The main tributaries of this river are Yang Ho in the north with a drainage area of 6,500 sq. mi. and Sang Chien Ho in the south with a drainage area of 10,800 sq. mi.; these unit at Chu Kuan Tun, flowing eastward, then to the southeast, and receive Kuei Shui Ho at Shih Chia Chai. The river gradually narrows down and enters "Kuan Ting gorge" in the south of Kuan Ting Tsun; its channel width varies from 300 ft. to 1,200 ft., the bed is steep and drops 1,100 ft. in a distance of 70 miles from Kuan Ting to San Chia Tien, the dividing point of upper and lower Yung Ting valley. Below San Chia Tien the river flows in a flat plain confined by levees; after coming down through the "triangular basin" it confluences with "North Canal". At Tientsin the river discharges into Hai Ho, the outlet of the principal streams in Hopei Province.

The average annual precipitation of Yung Ting valley varies from 14 to 30 inches, about 65-80% of which falls in summer; April and May are usually the dry months; in winter the precipitation falls in the form of snow and accumulates until late spring. The flood usually occurs in July or August, the melting of snow in spring or early summer although materially increases the discharge but seldom causes disastrous flood. Because of the steep slope and erodible land in "Upper Yung Ting" the river carries tremendous amount of silt during summer storms, the low water depending mainly on ground water supply contains but little silt. The minimum discharge at Lu Kou Chiao is about 170 c.f.s., the maximum flood peak may reach 346,000 c.f.s. Table 1 gives the drainage area and the flood discharges of the stations along Yung Ting Ho, Table 2 gives the average annual precipitations on Yung Ting valley stations.

Table 1

Flood Discharges, Yung Ting Ho
in thousands of c.f.s.

Station	Kuan Ting	San Chia Tien	Lu Kou Chiao
Drainage area, sq. mi.	17,500	18,150	18,160
Max. Flood	282.0	360.0	346.0
1924 Flood, July	203.0	184.0	173.0
1929 Flood, July	74.1	110.0	93.6
1929 Flood, Aug.	40.2	148.0	136.0

For hundreds of years the river has caused much trouble; it carries large amount of silt down to the lower part; as the slope is reduced, the velocity decreases; silt is deposited, lessening channel capacity; and during floods it may break the levees and finds a new way to the sea, causing great damages. The completion of levees together with some diversion works in eighteenth century although

Table 2

Average Annual Precipitation, Yung Ting Valley Stations

Station	Years of record	Precipitation June to Aug. % of total	Average Annual Precipitation in.
Ta Tung	7	66	13.82
Hun Yuen	7	63	14.10
Yang Kao	5	65	12.56
Tien Chen	3	74	15.12
Wei Hsien	6	67	16.61
Chang Cha Kou	10	75	13.59
Huai Lai	6	79	15.43
San Chia Tien	9	83	29.23
Lu Kou Chiao	11	81	22.12
Chin Men Cha	9	87	23.62
Shuang Ying	9	79	19.82

has succeeded in protecting Peiping (formerly the capital of China) from floods, the whole problem is not yet solved. The channel is again silted up, its capacity is not great enough even for moderate floods; the damages caused by 1924 and 1929 floods were considerable; the silt carried to and deposited at Hai Ho might amount to 8 million cu.yd. in 6 weeks, and decreases the navigable depth.

For the complete control of the river the North China River Commission has proposed the following projects

1. Kuan Ting and Tai Tsu Mu reservoirs for detension of flood flow from "Upper Yung Ting".

2. Reconstruction of diversion sluices at Lu Kou Chiao and Chin Men Cha.

3. Sand barriers in Yang Ho and Sang Chien Ho,

4. Desilting basins below Chin Men Cha.

5. Channel improvements.

<u>The Kuan Ting reservoir</u>. The Kuan Ting reservoir is situated just above the "Kuan Ting gorge", its capacity at spillway level is 11.5 billion cu.ft. with an average depth of about 25ft. The drainage area controled is 17,500 sq.mi., more than 90% of the Yung Ting water shed. The dam is 88.6 ft. high and is one of concrete, gravity, overflow type, its crest elevation is 1,528.5 ft. above sea level, the maximum flood level is 9.84' above spillway crest. The length of dam is 366.4 ft. on top, and the length of spillway is 295.3 ft.

It will be built of 1:3:6 concrete mixed with stone blocks, the surface finish will be 1:2:4 concrete about 6.ft. thick, other features may be seen from fig. 2.

On the bottom of dam there will be three sluices the upper part of each is a 19.7' semicircle and the lower part part is a 19.7'x4.9' rectangle with rounded corners as shown in fig. 3, on which the discharge curve is also shown.

Fig. 4 is the topographic map of the reservoir, fig. 5 is the area and capacity curves.

If the reservoir is not used for other purpose, it will be built as a detension basin; that is, there will be no controls on the sluices and the spillway. The behavior of the reservoir on floods may be seen from fig. 6 a, b, c, on which are shown the flood flow, outflow and storage curves of the reservoir. The retarding effect on floods is as follows:

Table 3

Effect of Kuan Ting Reservoir on Floods

Flood	Max. Inflow c.f.s.	Total Inflow bill. cu.ft.	Max. Outflow c.f.s.	Max. Storage bill. cu.ft.
Max.	282,000	23.62	81,900	11.10
1924	201,000	17.22	42,400	6.21
	141,000		39,200	3.78
	105,900		37,000	2.35
	70,600		34,200	1.22
1929	42,400	8.26	30,700	.37

Total volume of dam is estimated to be about 67,000 cu.yd., construction cost will be $440,000 (1,100,000 silver dollars, ($1=2.5 silver dollars approx.); land and property, $312,000; engineering contingency and administration fee, $225,000; total cost will be $977,000; cost per million cu.ft. of storage is $85.3.

<u>The Tai Tsu Mu Reservoir</u>

Although the drainage area of the portion between

Kuan Ting and San Chia Tien, the so-called Kuan Ting gorge, is only 550 sq.mi., due to its hilly and impervious character, the run-off is quite high; in the 1929 flood peak of 140,000 c.f.s., more than 100,000 c.f.s. came from this area; the Tai Tsu Mu reservoir about 37 mi. downstream from Kuan Ting is designed mainly to control the flood flow from Kuan Ting gorge, its capacity at level of spillway crest is 2.68 bill. cu. ft. with an average depth of 64.6 ft. The dam is 158.5 ft. high and is also of concrete, overflow gravity type, its crest is 984.3 ft. above sea level. The crest length of dam is 620.1 ft. and length of spillway is 492 ft. There are also three sluices on the bottom of dam the discharge curve of which is also shown in fig. 3.

Fig. 7 is the area and capacity curves of the reservoir and its behavior may be seen from fig. 8 a,b,c. The effect on floods is as follows:

Table 4

Effect of Tai Tsu Mu Reservoir on Floods

Flood	Max. inflow c.f.s.	Max. outflow c.f.s.	Max. discharge at San Chia Tien controled c.f.s.	uncontroled c.f.s.
Max.	152,000	106,100	131,000	360,000
1924	65,000	57,200	72,000	173,000
1929	109,100	51.400	79,000	136,000

Total volume of dam is about 209,400 cu. yd., total cost is estimated to be $1,770,000 or $660 per million cu.ft. of storage. The details of the reservoirs and other control works may be seen from the report of the Commission, 1933. (6)

C. Value of Yung Ting Water Power.

According to the Report submitted to world power conference sectional meeting 1933 by Chinese National Committee, the average investment per kw. capacity of electric utilities is about $240; of which $132 is the cost of generating equipment, $72 is the cost of transmission and distribution, and $36 represents the value of other tangible property. This is the result through analyzing 402 electric power plants which have about 50% of the total electric generating capacity in China. (7) The details may be seen from the following table:

Table 5

Investment per kw. Capacity in Electric Utilities, China
$1=2.5 silver dollars

Class	No. of plants	Aggregate capacity kw.	Investment	Investment per kw.
I (10,001 kw. up)	13	391,025	$97,148,000	$248
II(1001-10000 kw)	28	85,204	$15,863,000	$186
III(101-1001 kw.)	95	23,021	$ 5,829,000	$253
IV(100 kw. down)	266	7,350	$ 2,511,000	$342
Total	402	506,600		$240

No water power plant is included in the above analysis, ~~as~~ as there are only a little more than two thousand kw. developed; although China is said to be rich in water resouces.

The comparative higher per kw. cost of class I plants is due to the fact that most of these plants are situated in large cities and have higher distribution cost and more elaborate equipments than that of class II plants. Other features are tabulated as follows: (8)

Table 6

Distribution of Total Income in Electric Utilities, China

Class of plants	Distribution of total income(%) Fuel	Wages and salaries	Repairs, maintenance etc.	Depreciation	Interest, profit
I	25.0	14.0	17.3	9.3	34.4
II	31.5	16.8	18.0	11.1	22.6
III	27.0	20.5	25.6	10.0	16.9

Table 7

Operating Cost in Eletric Utilities, China

Class	Load factor	Capacity factor	Fuel consumption per kw.-hr. lb.	Fuel consumption per kw.-hr. ¢	Labor, maintenance per kw.hr.	Average price per kw.hr.
I	47.5	25.2	2.60	0.57	2.00	3.52
II	35.4	24.0	4.34	1.11	3.40	4.64
III	27.0	22.1			5.40	7.52

For class I plants, average cost of fuel = .57¢ per kw-hr.

= 25% of total income

Labor, maintenance and repair = 14+17.3=31.3 (%)

Cost of labor, maintenance, etc. per kw-hr= $.57 \times \frac{31.3}{25} = .72$¢

For class II plants, average cost of fuel= 1.11¢ per kw-hr.

= 31.5% of total

Labor, maintenance, repair, etc. per kw-hr.= $1.11 \times \frac{16.8+18}{31.5}$
= 1.23¢ per kw-hr.

The electric enterprise in Peiping, the greatest city in the vicinity of Kuan Ting reservoir, is also worthy of notice. There are two electric utility companies: The Peiping Electric Light Company has a total capacity of twenty thousand kilowatts; of which three thousand were installed before 1912 in its old plants; the new plant has a total capacity of 17,000 kw. which consists of 1-2,000 kw. generator installed in 1921, 1-5,000 kw. installed in 1923 and 1-10,000kw. installed in 1928; they are driven by steam turbines. After Nanking was adopted as the new capital of China the eletric market in Peiping rapidly declined, according to the report of 1931 (9) the annual output of 1930 is 30.9 million kw-hr.; peak load, 7,000 kw.; load factor, 50.4%; capacity factor, 20.5 %; the coal consumption is 6 lb. per kw-hr. max.,and 2.2 lb. min. with an average of 3 lb. per kw-hr. The selling prices are 8.8¢ per kw-hr. for lighting and 3.2-4¢ per kw-hr. for power; while the ordinary prices of class I plants are 7.2-8¢ per kw-hr. for lighting and 1.6-3.2¢ per kw-hr. for power.

The Peiping Electric Car Company has a total capacity of 3,000 kw., its power plant is situated at Tung Hsien about 15 miles from Peiping and consists of 2-750 and 1-1,500 kw. steam turbine generators. Under 33,000 volts the power is transmitted to Peiping for use on street cars; this company also sells its power for light and power purposes at Tung Hsien; and by some contracts the Peiping Electric Light Co. should buy some amount of the surplus power of this company on condition that the later shall not sell it directly to the consumers at Peiping. According to 1931 report the total output in 1930 is 6,930,000 kw-hr.; capacity factor, 26.4%; fuel cosumption, 3.46 lb. per kw-hr.

Although the two plants run at very low capacity factors the condition becomes better in recent few years, the annual output of 1930 is 10% greater than that of 1929, although no information concerning the present condition of these plants is at hand, it is believed that the increasing in demand is still going on.

To evaluate the water power it will be compared with the power produced in steam plant under the following conditions:

1. The power produced in an alternate steam power plant ~~at~~ at the proposed water power site. From table 5 the total cost per kw. investment of class II plant is $186, use the average value of 55% (7) as the cost of generating equipment, the cost per kw. of a steam plant without distribution and transmission facilities will be probably some what more than .55x186=$104; for safety sake a value of $100 per kw. is assumed.

The average cost of fuel is 1.11¢ per kw-hr. for class I plants, and .57¢ per kw-hr. for class II plants, some of the most efficient class II plants has coal efficiency of 2.4 lb. or .56¢ per kw-hr., it may be assumed that the fuel cost of a new plant will be .50¢ per kw-hr.

The cost of labor, maintenance, repair etc. is 1.23 ¢ per kw-hr. or \$25.8 per kw. per year for class II plants (based on average c. f. of 24%), and .72¢ per kw-hr. or \$15.8 per kw. per year for class I plants; a new plant of 5,000-10,000 kw. capacity will have a fixed operating cost of \$20 per kw. per year, assume 1/3 of this is the generating cost, a plant with generating equipment only will have a fixed operating cost of $\$20 \times \frac{1}{3} = \$6.7$ per kw. per year.

Assume:	cost of money	=	10 per cent
	taxes	=	1.5 " "
	depreciation	=	4.5 " "
	total fixed charges	=	16.0 " "

The cost per kw-hr. will be,

fixed chafges	= 100x.16 =		\$16 per kw. per year
fixed operating cost	=		\$ 6.7 " " "
total fixed cost	=		\$22.7 " " " "

A new plant may have a higher capacity factor and a value of .7 is assumed (this may be too large on the save side), hence

fixed cost at .7 c. f.= 22.7÷(8760x.7)	= .37¢ per kw-hr.
cost of fuel	= .50¢ " "
total cost per kw-hr.	= .87¢

2. The power generated by a new steam turbine generator.

in the plant of Peiping Electric Light Company. Assume the increment cost to be $80 per kw., **then** the cost of power will be:

fixed charges	= $80x.16 =	$12.8 per kw. per year
fixed operating cost	=	$ 6.7 " " " "
total fixed cost	=	$19.5 " " " "
fixed cost per kw-hr.	= 19.5 ÷ (8760x.7) =	.32¢
fuel cost " "	=	.50¢
total cost " "	=	.82¢

The hydro-electric plants will be probably located in the Kuan Ting gorge about 10-70 miles from this plant, assume a transmission cost of .06¢ per kw-hr. which is higher enough for such a distance; therefore,

cost of steam power per kw-hr.	= .82¢
cost of transmission " "	= .06¢
value of water power " "	= .76¢

3. Assume the water ~~water~~ power plant is installed to replace the old 5,000 kw. unit in the plant of Peiping Electric Light Company. The value of water power will be:

fixed charges less interest = $80x.06	= $4.8 per kw. per year	
fixed operating cost (assumed)	= $8.0 " " " "	
total fixed cost less cost of money	=$12.0 " " " "	
fixed cost per kw-hr. at .4 c.f.	= .37¢	(assumed)
cost of fuel per kw-hr. (about 3.1 lb)	= .72¢	(9)
	1.09¢	

total cost per kw-hr.	=	1.09¢
transmission cost per kw-hr.	=	.06
value of water power per kw-hr.	=	1.03¢

Based on these results it will be assumed that the value of Yung Ting water power is .75¢ per kw-hr. for primary and .35¢ per kw-hr, for secondary power.

D. The Combined Use of Kuan Ting Reservoir.

The silt problem. It may be seen that the cost of storage of Kuan Ting reservoir is much less than that of Tai Tsu Mu and that the capacity of the former can still be increased by raising the crest of the proposed dam. The fact that there are some very favorable power sites between Kuan Ting and Tai Tsu Mu also favors the combined use of Kuan Ting reservoir. There exist also some good power sites between Tai Tsu Mu and San Chia Tien, but seems to be no power possibility on further downstream. The question of developing water power is, now, limited to the operation of Kuan Ting reservoir and the construction of power plants in Kuan Ting gorge. The excceedingly high silting content of the river, which may be deposited and eliminate all the storage capacity, is worthy of notice. Through an extensive study on this problem the North China River Commission has found:

1. That the silt content of Yung Ting Ho varies widely with its discharge; 10-20% by wt. is very common during floods, the max. has reached 29% in July 18, 1929; while in extremely

low water the silt may amouunt to only few hundredth of one per cent.

2. That there is no definite relation between silt content and discharge quantity during floods, higher flood flow may carry less silt than lower ones; on the other hand the silt content during lower stage depends some what on the quantity of discharge as shown in the following table:

Table 8

Relation between Silt Content and Discharge, Yung Ting Ho

Discharge		Silt content Kuan Ting	Silt content San Chia Tien	Silt content Lu Kou Chiao
c.f.s.	c.m.s.	% by wt.	% by wt.	% by wt.
177	5	.25	.15	.07
353	10	.50	.30	.18
530	15	.80	.50	.40
706	20	1.10	.70	.60
1,060	30	1.70	1.10	.90
1,412	40	2.10	1.70	1.20
1,765	50	2.60	2.50	1.70

3. That from the discharge and silt content relations at various stations the average monthly ~~monthly~~ silt content is estimated as shown in table 9., the total yearly silt content is about 907 million cu.ft. Also, according to the survey on "triangular basin" (see p. 4) in 1921 and 1927, the average yearly silt deposit is about 884 million cu.ft.; with the

addition of the silt carried by 1924 flood to Ta Tsing Ho and also the fine silt carried through the "triangular basin" to sea, the average yearly silt content of the river is estimated to lie between 986 and 1,240 million cu.ft.

Table 9

Monthly Silt Content, Yung Ting Ho

Month	Silt content % by wt.	Total amount mill. cu. ft.
J	.....	.35
F	.....	.39
M	.91	29.80
A	.82	11.48
M	.95	5.69
J	1.73	13.45
J	13.40	449.00
A	8.55	380.00
S	.42	4.45
O	.56	6.57
N	.40	4.77
D	.07	.70
		906.65

For flood control only, the Kuan Ting reservoir is designed as a retarding basin; in usual times it does not hold water, the river will flow within the original channel and there will be no opportunity for silt deposition; the flood flow is held but temporarily, only part of its silt content will be left in the reservoir. For storage or power purpose the presence of sluggish water makes silting unavoidable, and the whole reservoir will be quickly filled if the total amount of silt is allowed to settle. Fortunately more than 90% of silt is contained in the flood flow of July and August, in this period if the water level of the reservoir is kept very low so that the river flows within its channel and the sluice gates are properly controled, the silt deposit may be greatly reduced. Thus the combined use of Kuan Ting reservoir may be attained in the following manner: raise the dam crest to level 1533(4.11 ft. higher), the capacity of the reservoir will be increased to 13.56 bill. cu.ft.; during July and Aug. the water level will be kept at or below 1492, the reservoir will act practically as a detension basin; the average yearly deposit will, according to the Report of the Commission, 30 million cu.ft. In other times the reservoir is used mainly for storage purpose and 100 per cent deposition is assumed; since nearly all silt of Yung Ting Ho comes from the region above Kuan Ting the values given in table 9 are used; the sum of ten months (Sspt.-June) is 77.7 million cu.ft., the total yearly silt deposit will be 107.7 million cu.ft. on the average.

In the Report, 30% of reservoir capacity is allowed for silt deposition, original capacity of reservoir is 11.5

bill. cu. ft.; with 30% of silt its capacity is reduced to 11.5x.7 = 8.05 billion cu.ft.; capacity of reservoir when raised to level 1533 is 13.46 bill. cu. ft., and the excess capacity is 13.46-8.05 = 5.41 bill. cu. ft.; therefore the reservoir can be used 5.41÷.1077 = 50 years for combined purposes.

The cost of increasing the height of dam 4.11 ft. is estimated to be $92,000 (6); cost of sluice gates at $100 per sq. ft. is $74,600; with some contingency, the total cost of storage is roughly $180,000.

<u>The regulation and utilization of flow</u>. The monthly discharges at Kuan Ting 1925-1934 are shown in table 10, since the reservoir will be kept at elev. 1492 in July and August, the storage capacity is limited to 795 million cu. ft. or 300 c.f.s. months. In other times the reservoir will be used mainly for power and the storage is only limited by its capacity. By this way the dependable part of the regulated flow is found to be 440 c.f.s. (table 11).

The limit to which the discharge of the river can be utilized may be estimated as follows: assume the increment cost of water power to be $50 per hp.; annual fixed and operating cost, 14%; the yearly cost per hp. of increment capacity will be $7.0; assume value of secondary power less transmission cost to be .35¢ per kw-hr., the yearly output per hp. should be not less than 7.0÷.0035 = 2,000 kw-hr.; with generator efficiency of 94% the yearly output per hp. is .746x.94x8,760 =6,150 kw-hr., if the capacity factor is 100%; so that it will be economical if the increment capacity has a c.f. of 2,000÷6,150 = .325; that is, the flow available 32.5% of time can be utilized.

Since part of the the discharge data are prepared from rating curves and there may be considerable error, also the duration curve of monthly discharge (fig. 9) gives higher availableflow than true daily condition; the wheel capacity of the plants will be assumed to be 1,200 c.f.s. which is the flow available 50% of time according to fig. 9. The discharage from the area between Kuan Ting and the power plant is negligible compared with that from above Kuan Ting, a drainage-area of 17,500 sq. mi.

From Sept. to June, the river never reaches a very high stage, the max. occured on March, 1925 corresponding to a discharge of 23,000 c.f.s. which is but one half of the discharge capacity of the sluices with water level at spillway crest; it seems that the whole reservoir may be used for power in this period; in fact, with storage limited to 795 million cu. ft. in July and August and a wheel capacity of 1,200 c.f.s., a storage of 2,000 c.f.s. months or 5.26 bill. cu.ft. for power is found to be ample; leaving 13.56-5.26 = 8.30 bill. cu.ft. for flood. Hence the reservoir may be operated according to the following rule: in each month, first the dependable flow of 440 c.f.s. is maintained, the surplus water will be stored; it seems sufficient to have 1,500 c.f.s. months to furnish primary power

Table 10

Monthly Flow in c.f.s. at Kuan Ting Station, 1925-1934
drainage area = 17,500 sq. mi.

	1925	1926	1927	1928	1929	1930	1931	1932	1933	1934
J	826	2424	5800	883	706	2740	735	2510	1592	1080
F	826	5850	7310	1172	706	2940	805	3408	2780	3020
M	1871	5110	8400	2180	1613	1620	1543	2886	5015	3180
A	1553	1536	1598	876	755	314	459	886	1635	2120
M	1012	1084	745	247	106	109	275	318	523	735
J	1828	2060	950	7060	268	307	304	752	3510	1433
J	2170	1640	1242	2038	4240	700	2490	3766	4040	6690
A	2630	1117	2296	1960	4200	332	1340	2390	3320	5520
S	2240	1190	1994	993	675	460	1100	1808	1695	5150
O	1830	1151	1830	1778	644	414	706	1757	1585	5130
N	1390	127	1580	1860	463	381	798	1507	894	3550
D	939	1080	1052	1453	713	339	706	992	823	2430

Note: The monthly discharges of 1925 are determined from the hydro-graph fig. 45, Report of the Commission, except those of Jan., Feb. and Dec. which are determined from the water level and rating curves, 1925. Those of 1926-1928 are also determined from the corresponding water level curves and rating curve of 1925. From 1929-1931 the discharges are determined from the hydrographs of these years; the data of Jan., Feb. and March are lacking, a value of 706 c.f.s. (the lowest monthly discharge in winter, 1925-1934) is assumed for Jan. and

Feb., the discharge of March are determined by comparing with the records of San Chia Tien and Lu Kou Chiao. (Report fig. 16) From 1932 to 1934 the discharges are determined from the water level curves and rating curve of 1930.

Table 11

Regulated Monthly Flow in c.f.s. at Kuan Ting Station
1925-1934

drainage area = 17,500 sq. mi., storage in reservoir for power = 5.26 bill. cu. ft. from Sept. to June, and .795 bill. cu. ft. in July and August.

	1925	1926	1927	1928	1929	1930	1931	1932	1933	1934
J	826	2163	5300	1200	1200	1775	440	2010	1384	1080
F	826	5850	7310	1200	712	2940	440	3408	2780	2520
M	1371	5110	8400	1687	1200	1620	729	2886	5015	3180
A	1553	1536	1598	1200	1200	1200	859	1200	1635	2120
M	1512	1584	1245	1200	874	523	675	1200	1200	1235
J	3028	3260	2150	7483	668	707	704	1256	4533	2633
J	2170	1640	1242	2038	4240	700	2490	3766	4040	6690
A	2630	1117	2296	1960	4200	440	1340	2390	3320	5520
S	1040	440	794	440	440	440	440	608	495	3450
O	1330	701	1330	1131	440	440	440	1257	1200	5130
N	1390	440	1580	1360	440	440	524	1507	1200	3550
D	1200	767	1200	1453	440	440	706	1200	902	2430

(this is designated as "primary ~~discharge~~ storage") in the following probable dry months; as the storage capacity is limited to 300 c.f.s. months (795 mill. cu. ft.) from the end of June to end of August, the "primary storage" at the end of May will be 700 c.f.s.-months; and 1,100 c.f.s.-months at the end of April. After the "primary storage" has been provided, the surplus water will be used for development of power until the wheel capacity 1,200 c.f.s. is attained; if there is still some water left, it will be stored in the reservoir with a limit of 200 c.f.s.-months (5.26 bill. cu. ft.). The monthly plant and total (plant + waste) discharges are thus computed and tabulated as shown in table 11, the duration curve of the regulated flow is also shown in fig. 9.

II. The Economy of Yung Ting Power Development----Preliminary Investigation.

Nomenclature

h = height of crest of spillway, in ft.

H = max. flood level above crest of spillway, in ft.

h' = height of flashboards or control gates on crest of spillway. $h+h'$ will be the available head of the dam.

Y_s = volume of dam in cu. yd. per ft. of spillway = $\frac{h^2+2Hh}{66}$ (Barrows, p. 307).

Y_A = volume in cu. yd. per ft. of the abutement section = $\frac{(h+H)^2}{66}$ the volume per ft. of dam will be $\frac{(h+h')^2}{66}$ for an available head of $h+h'$, if there is no spillway for flood flow.

L = equivalent length of dam = total volume $\div$ max. cross-section, and lies between the crest length of dam and the channel width.

c_f = cost in dollars per sq. ft. of flashboard.

c_g = cost in dollars per sq. ft. of gate.

c_c = cost in dollars per cu. yd. of cyclopean concrete.

q = discharge per ft. of spillway = $3.94H^{\frac{3}{2}}$.

V = velocity in ft. per sec. = $.85\sqrt{2gh}$ for flow through sluice.

e = efficiency of turbine or generator.

Fig. 10 is a topographic map of Yung Ting Ho between Kuan Ting and San Chia Tien, the profile and cross-sections are shown in fig. 11 and fig. 12 respectively. As the channel is narrow, varying from 200 ft. to 500 ft., and is the flood flow is comparatively high; when dams are built across the river, the method of disposing flood is worthy of notice. This may be (1) spillway with temporary flashboards, (2) spillway with stanchion flashboards or crest gates, (3) sluice gates, (4) diversion channel or (5) tunnel or siphon spillway.

<u>The spillway with temporary flashboards.</u> Suppose the height of temporary flashboards is limited to 5 ft. (h'); for a dam of h ft. high, the available head is h+5, the volume of the corresponding abutement is $Y_A = \frac{(h+5)^2}{66}$ in cu. yd. per ft. of dam (see p. 26), the volume of the spillway section is $Y_S = \frac{h^2+2hH}{66}$ in cu. yd. per ft. of dam, where H is the height of max. flood over spillway crest, the excess volume is $Y_S - Y_A$ $= \frac{(h^2+2hH) - (h+5)^2}{66} = \frac{2hH-10h-25}{66}$ per ft. of dam, and the cost increased will be $c_c(Y_S - Y_A) = \frac{7}{66}(2hH-10h-25)$; cost of flashboaeds is $c_f \times h' = 3 \times 5 = \$15$ per ft. of dam. Therefore, the excess cost per ft. of dam is $15 + \frac{7}{66}(2hH-10h-25)$ in dollars, $= 12.35 + 0.212(h)(H-5)$ in dollars.

q = max. discharge in c.f.s. per ft. of dam = $3.94H^{\frac{3}{2}}$, therefore cost per c.f.s. flood discharge is $\frac{0.212h(H-5)+12.35}{3.94H^{\frac{3}{2}}}$; substituting with various values of h and H, the per-c.f.s. costs are obtained as shown in table 12.

It might be considered from the above formula that the cost per c.f.s. decreases with the increase of H and the use of higher H will be more economical; in fact, as H increases the volume of the abutement section $\frac{(h+H)^2}{66}$ for the same available head h + 5 also increases, this will counterbalance the saving in the spillway section. H is usually governed by the quantity of flood and the width of channel, and this table can be used only for comparison with other methods.

Table 12

Additional Cost of Dam and Control Works per c.f.s. of Flood Discharge-----Spillway with Temporary Flashboards, $h' = 5$

Available head $h + h'$	40	60	80	100	120
H = 10, q = 124.5	\$.397	.566	.739	.910	1.070
H = 15, q = 228.3	\$.370	.565	.750	.935	1.130
H = 20, q = 351.0	\$.363	.532	.715	.896	1.076
H = 25, q =492.5	\$.326	.500	.671	.845	1.020
H = 30, q = 646.0	\$.306	.470	.635	.800	.965

<u>The spillway with stanchion flashboards or crest gates</u>

Limiting the height of stanchion flashboards to 10 ft. and that of crest gates to 20 ft. Assume cost of stanchion flashboards = \$5 per sq. ft. and crest gates = \$17.5 per sq. ft. including control works, the per c.f.s. costs with various h, h' and H are calculated and tabulated as shown in Table 13.

Available head $= h + h'$

Volume of dam $= \dfrac{h^2+2hH}{66}$ in cu. yd. per ft.

Volume of corresponding abutement $= \dfrac{(h+h')^2}{66}$ in cu. yd. per ft.

Increasing or decreasing in volume $= \dfrac{h^2+2hH}{66} - \dfrac{(h+h')^2}{66}$

$$= \frac{2hH-2hh'-h'^2}{66}$$

Cost of flashboards or crest gates = ch' per ft. of dam,

Total cost for flood disposal = $ch' + c_c \frac{2hH-2hh'-H'^2}{66}$ in dollars/ft.

q = max. discharge per ft. $= 3.94H^{\frac{3}{2}}$ in c.f.s.

Cost per c.f.s. $= (ch' + c_c \frac{2hH-2hh'-H'^2}{66}) \div 3.94H^{\frac{3}{2}}$

Table 13

Additional Cost of Dam and Control Works per c.f.s. of Flood Discharge----- Spillway with Stanchion Flash-boards or Crest Gates

Available head $h + h'$, ft.	40	60	80	100	120	Remarks
$H = 10 = h'$ $q = 124.5$	\$0.316	0.316	0.316	0.316	0.316	Stanchions
$H = 15$, $h' = 10$ $q = 228.3$	0.312	0.400	0.497	0.629	0.680	Stanchions
$H = 15$, $h' = 15$ $q = 228.3$	1.046	1.046	1.046	1.046	1.046	Crest gates
$H = 20$, $h' = 20$ $q = 351$	0.875	0.875	0.875	0.875	0.875	Crest gates
$H = 25$, $h' = 20$ $q = 492.5$	0.668	0.710	0.753	0.796	0.839	Crest gates
$H = 30$, $h' = 20$ $q = 646$	0.540	0.606	0.672	0.737	0.804	Crest gates

<u>Sluice</u>. For simplicity, assume cost of sluice gates and controls = \$100 per sq. ft. of sluice and does not vary with head, since V, the velocity of water flowing through sluice, $= C.85\sqrt{2gh}$; this will also equal to q, discharge per

sq. ft. of sluice, where h is the head acting on the sluice and is approximately equal to the height of dam; then the cost per c.f.s. will be equal to $100/0.85\sqrt{2gh}$; substituting with various values of h the results are tabulated as follows:

Table 14

Cost per c.f.s. of flood discharge --- Sluice gates and controls

Head(h) ft.	60	80	100	120
Cost per c.f.s.	$1.88	1.63	1.45	1.33

The use of flood diversion channel seems not practicable on this portion of the river, the siphons are subject to clogging and also might not be economical, they are left out of consideration.

It may seen from tables 12 - 14 that the cost per c.f.s. is highest when sluices are used, although the assumed unit cost of $100 per sq. ft. may be too high for gates under 60 ft. head or less; so that the spillway will be the only route of flood flow; which varies from 83,000 c.f.s. at Kuan Ting to 152,000 c.f.s. at Tai Tsu Mu and from 106,000 c.f.s., the outflow of Tai Tsu Mu reservoir to 131,000 c.f.s. at San Chis Tien; the available length of spillway which is limited by the width of channel will be from 200 ft. to 400 ft. ; therefore H, the height of max. flood over spill-way crest will be about 20 - 25 ft., and the use of crest gates to increase the available head is economical only when the dam is higher than 100 ft.. These results are, however, obtained from very rough estimations and approximations, and can be used only as a general guidance.

Cost of Dam In order to estimate the cost of dam, assume they will be of concrete overflow gravity type, the volume per ft. of dam is $Y_s = \frac{h^2+2hH}{66}$, suppose the depth of foundation in 30 ft. which is the general condition on this section of Yung Ting Ho, and as the base width of dam is approximately 0.7(H+h), (10) the total volume of dam in cu. yd. ~~per ft.~~ is equal to $L\left(\frac{h^2+2hH}{66} + \frac{21(h+H)}{66}\right)$ where L is the "equivalent length of dam" (see page 25). The available head is h and the undeveloped water power will be $\frac{Qwhe}{550} = \frac{Qhx62.5x8.8}{550} = \frac{Qh}{10}$, hence cost of concrete in dam per hp. is equal to

$$\frac{10Lc}{Q}\left(\frac{h^2+2hH}{66} + \frac{21(h+H)}{27}\right),$$

when c = cost of concrete per cu. yd. = \$7

Q = available flow of the river = 1,200 c.f.s.

assume H = 20 ft.,

substituting and simplifying, cost of concrete in dam per hp.

$= \frac{7}{7920}L\left(\frac{h^2+91.4h + 1,028}{h}\right)$; substituting with various values of L and h, the results are obtained as shown in table 15.

Again, neglect the fact that L increases somewhat with h, and differentiate

$$\frac{d}{dh}\left(\frac{7L}{7920}\right)\frac{h^2+ 91.4h+ 1,028}{h} = 0$$

if L = constant,

$$h^2 - 1028 = 0$$

so that h = 32.1 ft. which is the most economical h for the

Table 15

Cost of concrete in dollors per hp. --- concrete overflow gravity dams with Q = 1,200 c.f.s.

h	40'	60'	80'	100'	120'
L = 200'	27.8	29.8	32.6	35.6	38.8
L = 300'	41.7	44.7	48.9	53.4	58.2
L = 400'	55.6	59.6	65.2	71.2	77.6
L = 500'	69.5	74.5	81.5	89.0	97.0

concrete used in dam per hp. Since the cost of cofferdam, excavation etc. is but little effected and the cost of power house and equipment decreases with the increase of h, while the land and property flooded are greater for a single high dam than two separate dams with the same total height; the most economical height of dam, as a whole, will be 40-50 ft., if conditions such as foundation, equivalent length of dam etc. are the same throughtout this portion of Yung Ting Ho; as a matter of fact, this is far from being frue. Refering to table 15, it may be seen that the cost per hp. is increased only 10 % when h is raised 20 ft., while the equivalent length of dam has much greater influence; the cost per hp. for a dam 120 ft. high with L = 200' , is less than that of a 40'-dam with L= 300';

<u>The limiting height of dam.</u> The limiting height of dam, also, may be roughly determined; the total cost of concrete in

dollars is equal to $\frac{7}{66}(h^2+91.4h+1028)$, the increment cost per ft. of h is equal to $\frac{d}{dh}(\frac{7}{66})(h^2+91.4h+1,028) = \frac{7}{66}L(2h+91.4)$, as hp. = Qhwe/ 550 = 120h, the increment cost per hp. is, therefore, equal to $\frac{7}{7920}L(2h+91.4)$,substituting with various values of L and H, the results are tabulated as shown in Table 16:

Table 16

Increment cost of concrete per hp. --- concrete overflow gravity dams

h	40	60	80	100	120	140
L=200'	$30.4	37.4	44.5	51.5	58.6	65.6
L=300'	45.5	56.0	66.6	77.2	87.9	98.5
L=400'	60.8	74.8	89.0	103.0	117.2	131.2
L=500'	75.9	93.4	111.1	128.7	146.5	164.1

From the monthly flow duration curves the average available flow is 1,010 c.f.s. and the capacity factor of the power plant will be 1,010 ÷ 1,200 = .85, for safety sake a value of .7 is assumed and the annual output per hp. with generator eff. = .94, will be .7 x 8,760 x .746 x .94 = 4,310 kw-hr.; of which about 50 % will be primary; assume a value of .75 ¢ per kw-hr. for primary power and .35 ¢ for secondary, the annual income will be .0055 x 4,310 = $23.8; capitalized at 14 %, this becomes $170, which is the limit to the investment per hp.; leaving $40 per hp. for the cost of power house and equipment, the cost of dam and waterway will be limited to $130; as the cost of concrete is around 80-90 % of the increment cost of dam, the increment cost of concrete is therefore limited to \$110-130, and therefore the height of dam is limited to 100-160 ft. according to the conditions of site. This, also, may be seen from the other point of view, along the section from Kuan Ting to San Chia Tien a dam site with L less than 200 ft. is very rare and sites with L = 400-500 ft. are easily found, if a dam is carried to the height, say, above 180 ft., the cost of concrete for lsat 40 ft. (140-180) will be more than \$70 per hp. ; a 40 ft. dam with L = 400-500 ft. has a cost around \$60 per hp., so that the building of two separate dams will be more economical; this also limits the height of dam to 140-160 ft.

Waterway. By the same way the limitation to the construction of waterway may be determined also. It has been found that the economic size of reinforced concrete penstock is 12 ft. dia. (p. 54), the cost per ft. is about \$29; with the addition of the cost of excavation, anchorages, surge tank etc., the cost will be roughly \$40 per ft. or \$40,000 per thousand ft.; to compare with the cost of dam at \$60-80 per hp. or \$7,200-9,600 per ft. of head (for Q=1,200), and because of the fact that cost of power house and equipment is lower under higher head; the use of reinforced concrete pipe is justifiable when the head gained per thousand ft. of pipe is more than 3-4 ft. The economic size of concrete lined rock tunnel is about 100 sq.ft. (about \$70/ft.), and that of steel pipe is around 10 ft. diameter, they are also limited by the cost of \$12,000 for

1 ft. gain of head.

It may be seen from the topographic conditions and economic considerations that a number of plants are required to develope all the water power resources between Kuan Ting and San Chia Tien; the dams will be generally 40'-60' high, and on very good sites they may be carried to 100' or more, other factors such as the construction of penstock and tunnel also have some effect on the height and location of dams. Only dams of concrete overflow gravity type have been considered, other types such as hollow, arch and earth are left to further investigation. On a divided-fall plant the cost of waterway shall not exceed $12,000 for 1 ft. of head gained by its construction. To dispose the flood flow the use of sluice is not economical, for dams higher than 100 ft. crest gates may be used, and for low dams a spillway with temporary flashboards would be more economical. The cost of dam and waterway will be $50-100, with the addition of the cost of power house and equipment, the total cost per hp. will be $100-150.

III. Cost and Value of Yung Ting Power Development.

A. The Power System.

With the forgoing considerations in view it seems that the water power development in Kuan Ting gorge may be arranged as shown in table 17, the positions of these plants may be seen from figs 10, 13 and 14.

Table 17

Power Plants---Kuan Ting Gorge Development

Plant No.	1	2	3	4	5	6
Location of dam	An Chia Hsuan, (CC)	Near Lao Hu Chia, (EE)	Chu Wo, (HH)	Near Ho Nan Tai, (KK)	Near Lung Chia Tsun	Near San Chia Tien
Location of P.H.	"	Near Yen Ho Chen, (FF)	"	Near Tung Ping Wang Tsun, (MM)	"	"
Ht. of dam.	42'	100'	60'	70'	60'	70'
Ht. of flash-boards or crest gates	5'	20'	5'		5'	5'
Length of waterway		5000'		30000'		
Normal head water elev.	1420	1321	1129	798	533	403
Normal tail water elev.	1375	1158	1066	571	470	330
Normal eff. head, ft.	45	153	63	187	63	73
Plant capacity, hp.	5400	18360	7560	22440	7560	8760

An alternat~~e~~ive plan for plants No. 1, 2 and 3 may be seen from table 17a.

Table 17a

Alternative Plan---Kuan Ting Gorge Development.

Plant No.	1a	2a	3a
Location of dam	About 12,000 ft. downstream of Yu Chou, (DD)	Near Lao Hu Chia, same as No. 2	About 5,000 ft. upstream of Chu Wo, (HH)
Location of P.H.	"	"	"
Ht. of dam, ft.	100	60	80
Ht. of flash-boards or crest gates, ft.	20	5	5
Normal head water elev.	1414	1268	1169
Normal tail water elev.	1296	1205	1086
Normal eff. head, ft.	118	63	83
Plant capa-city, hp.	14160	7560	9960

Out of the total drop 1,100 ft. from Kuan Ting to San Chia Tien; about 140 ft. are used for detension of flood in Tai Tsu Mu reservoir; 584 ft. are effective (based on table 17) for the development of power, which amounts to 67,920 hp. with Q = 1200 c.f.s.; the rest are reserved for back water and friction loss in the waterway. The topographic condition between Tung Ping Wang Tsun and San Chia Tien is not known, and two concentrated-falls are assumed temporarily. As a closer approximation, the general arrangement of No. 2 plant and that of No. 3 are assumed and the cost estimated.

B. Cost and Value of Plant No. 2.

Plant No. 2 is a divided-fall, its general arrangement may be seen from figs. 15, 16 and 17. The dam is 100 ft. high and located 700 ft. downstream from Lao Hu Chia. It is one of concrete overflow gravity type, its length at spillway crest is 362 ft., and its length at river bed is 205 ft. The spillway crest will be at elevation 1303, with 12 taintor gates 20' x 20' each, operated by two 25-ton traveling hoists. The two intake gates are of Stoney type with 7' x 12' openings, and are operated by a 8-ton traveling hoist. The waterway consists of 340' reinforce concrete pipes of 12' diameter, 4000' concrete lined rock tunnel with 100 sq. ft. water area, 320' steel pipes of 12' diameter and 3-300' steel branch pipes of 8 ft. diameter. The surge tank is 32 ft. diameter, 75 ft. high and is located a short distance beyond the downstream end of tunnel.

The power house is located about 3000' downstream of Yen Ho Cheng, it is designed as one of single floor type, 90 ft. long and 50' wide. On the main floor are three units of vertical Francis turbine with 5750 kva. generator and direct connected exciters operating at 300 R.P.M. Each turbine has a capacity of 6600hp. under 153' head with specific speed N_s 45.2. The distance center line to center line of units is 22ft.

The auxiliary bay consists of two stories, on the first floor are the bus compartment, auxiliary transformers, machine shop and storage room. The switch board, switches, battery room and the offices are on the second floor, which is 14' above the first. The high tension yard will be located outside the power house. The cost of the plant are estimated to be:

Dam	$850,000
Intake and waterway	$400,000
Power house	$158,000
Equipment	$515,000
Total	$1923,000

adding 20 %, of which 15 % is the allowed for engineering contingency and 5% for interest during construction, this becomes

cost + contingency and interest	=	$2,310,000	@ $117 per hp.
cost of storage	=	180,000	@ $ 9 per hp.
total	=	2,490,000	@ $126 per hp.

The annual cost of this plant will be; fixed changes

1. Interest = 10 %
2. Depreciation and taxes = 2.5 %

total annual fixed charges will be 2,490,000 x 12.5

= $ 312,000

Operating cost

wages and salaries	$ 14,000
maintainance and repair	$ 4,000
	$ 18,000

Total annual fixed and operating cost = \$ 330,000

Annual output

assume capacity factor = .7 (see page 33)

efficiency of generator= .94

annual out-put = 19,800 x .7 x .94 x 8760

= 85,400,000 kw-hr.

cost per kw-hr = 330,000 ÷ 85,400,000

= \$ .00386 or .386 ¢.

assume value of power = .55 ¢ per kw-hr.

total annual income = .0055 x 85,400,000

= \$ 470,000

annual profit = 470,000 - 330,000

= \$ 140,000

capitalized at 10 %

140,000 ÷ .10 = \$ 1400,000 which represent the value of power site.

Effect of storage

Without storage, the dependable flow is 107 c.f.s.

average available flow = 840 c.f.s. (assumed)

The power output is about 1/8 primary and 7/8 secondary; and value of power will be .35 x 7/8 + .75 x 1/8 = .40 ¢ per kw.-hr.

Total investment without cost of storage = \$ 2,310,000

fixed charges 12,5 %	= \$ 289,000
operating cost	= 18,000
Total	\$ 307,000

total annual output = 85,400,000 kw.-hr.

cost = .360 ¢ per kw.

Total annual income at .4000 ¢ per kw.-hr.

= $ 342,000

annual profit = 342,000 - 307,000 = 35,000

This, capitalized at 10 % = 35,000 ÷ .10 = $ 350,000 value of site

Value of site with storage	=	$ 1400,000
Value of site without. "	=	350,000
Value of storage		1050,000
Cost of storage		180,000
Net value of storage for plant 2.	=	$ 870,000

C. Cost and value of plant No. 3

Plant No. 3 is a concentrated fall, and is located at Chu-Wo. The general arrangement is shown in Figs. 18 and 19. The dam is one of concrete gravity type, its length at crest of spillway is about 500 ft. The max. flood elev. is 1146 which is 20' above spillway crest, the flashboard is 5' high and the normal water elevation is 1129. The length of spillway is 350 ft. with H = 20 ft., the discharge capacity is 123,000 c.f.s. The power house is located on the left of spillway, it is designed also as a single floor type. On the main floor are the three units which are vertical Francis turbines with 2250 kva. generators, and direct connected exciters.

The normal speed is 240;, head, 63 ft.; and the specific speed , N_s, of turbines is 69.7. The distance center line to center line of units is assumed as 23.0 ft., and the crane capacity is 40 tons. The auxiliary bay will be in consistent with the general shape of abutement section; the first compartment consists of two stories, on the first story are the bus compartment, to auxiliary transformers and the machine shop. The switchboard, switches and offices are on the second story, from which a stairway leads to the entrance to second compartment. On the latter are the battery room, the storage room, and also the stairway to intake floor. The high tension yard is on the outside of power house. The cost of plant 3 is estimated to be (see page 67)

Dam (including power house superstructure)	$ 695,000
Power house superstructure	43,400
Equipment @ $ 32 per hp.	255,600
Total	$ 994,000

adding 20 % , of which 15 % is allowed for engineering contingency and 5 % for interest during construction.

cost + contingency and interest = $ 1,193,000 a $150 per hp.

cost of storage = $180,000 a $22.6 per hp.

Total cost = $ 1,373,000 @ $ 172 per hp.

The annual cost of this plant will be:fixed charges 12.5 %

1,373,000 x .125 = 172,000

operating cost

wages and salaries = 10,000

maintenance and repairs = 3,000

total annual cost = \$185,000

annual output

assume capacity factor = .7

efficiency of generator = .94

annual output = 7980 x .746 x .7 x .94 x 8760

= 34,400,000kw.-hr.

cost per kw.-hr. = 185,000 ÷ 34,400,000

= .00537 or .537 ¢.

assume the value of power = .55¢ per kw.-hr.

Total annual income = .0055 x 34,400,000 = \$189,000

and the annual profit will be 189,000 - 185,000

= \$ 4,000.

This capitalized at 10 %

4,000 ÷ .10 = 40,000 which represents the value of power site.

D. Conclusion and suggestions

With the value of power at .55¢ per kw.-hr. the development of power in Kuan Ting Gorge is found to be economical; the cost of power at plant No. 2 is estimated to be .386¢ per kw.-hr. and that of plant No. 3 is .537¢ ; for other plants the cost will not deviate very much from these limits.

The total available head on the section between Kuan-Ting and San-Chia Tien is about 600 ft. allowing 130-140ft. for detension of flood in Tai Tsu Mu reservoir; with Q = 1,200 c.f.s. the total power will be 72,000 hp. which will be divided among about 6 plants; the cost per hp. will vary from $100 to 150.

The capacity of Kuan Ting reservoir used for power development is 5.26 bill. cu. ft. or 2000 c.f.s. months from Sept. to June and .795 bill. cu. ft. or 300 c.f.s. months in July and August; the additional expense is but $180,000 when combined with the proposed Kuan Ting reservoir. If the reservoir is used only for power, with 5.26 bill. cu.ft. storage capacity the dam will be 73 ft. high and the total cost will be $700000 to $800000 including cost of outlet works. The maximum flood will be 280,000 - 360,000 c.f.s.; that is, it will be 200,000 c.f.s. higher, if not controlled by the reservoir. As may be seen from tables 12 and 13, the handling of flood costs more than $ .3 per c.f.s., the construction of each dam will cost $60,000 more. Take the case of plant No. 2, the value of site or annual profit capitalized at 10 % is $ 1,400,000. While the cost of Kuan Ting reservoir for flood and power is only $1,150,000; thus the profit from power development will carry the expense of flood control more than enough. If the whole power system can be developed very quickly, it is economical to provide some flood surcharge on each dam so that Tai Tsu

Mu detension reservoir may be omitted,but this is not practicable as theree is no market for such amount of power at present.

Without the storage in Kuan Ting reservoir the min. monthly flow in 107 c.f.s. and the power output will be mostly of secondary nature and its value will be reduced to .4¢ per kw.-hr., in this case some very favorable power site such as plant No. 2 can still be developed with profit.

The cost of power has been estimated very roughly but safely, there may be considerable savings; very high unit prices of material and labor have been assumed, the interest rate is also high. the dams are all of concrete gravity type, other types especially arch and hollow dams may be found more economical. Rock tunnel are assumed as the water way of plant No. 2, while cheaper penstocks may be adopted if topographic conditions favor their use. The depth of foundation is assumed to be 30 ft. in all cases, on very favorable sites this may be much less and the cost of dam will be lower than that assumed. The discharge between Kuan Ting and San Chia Tien has be neglected, this may be considerable when the plant is located far downstream of reservoir, and hence higher capacity units can be used with low increment cost.

All the silt content Sept. to June is assumed to be deposited in the reservoir and the deposition of 4 % by weight of flood storage is assumed for J uly and August. The correctness of these assumptions should be determined by further experimental works.

According to the present condition of electric enterprise in Peiping, it is considered that a 4,000 - 6,000 kw. hydro-plant may be built with profit presently or in the near future, more detailed analysis of the power demand and operating condition ~~are~~ is, of course, necessary. As the first plant can be located on the best site, the cost per hp. will be around $120 including cost of storage and the cost of power will be less than .4¢ per kw.-hr.

Use of Hydrograph

dependable flow = 440 c.f.s.

"primary storage = 1,500 c.f.s. months, Sept. to March

= 1,100c.f.s. months, at the end of April

= 700 c.f.s. months at the end of May

= 300 c.f.s. months at the end of June

= 300 c.f.s. months in July and August

maximum storage = 2,000 c.f.s. months Sept. to end of April

= 1,500 c.f.s. months at the end of May

= 300 c.f.s. end of June to end of August

	Month	Natural flow	Plant flow	Storage	Waste	Total storage	Total outflow
1925	J	826	826	0	0	1500	826
	F	826	826	0	0	1500	826
	M	1871	1200	500	171	2000	1371
	A	1553	1200	0	353	2000	1553
	M	1012	1200	-500	312	1500	1512
	J	1828	1200	-1200	1828	300	3028
	J	2170	1200	0	970	300	2170
	A	2630	1200	0	1430	300	2630
	S	2240	1040	1200	0	1500	1040
	O	1830	1200	500	130	2000	1330
	N	1390	1200	0	190	2000	1390
	D	939	1200	-261	0	1739	1200
1926	J	2424	1200	261	963	2000	2163
	F	5850	1200	0	4650	2000	5850
	M	5110	1200	0	3910	2000	5110
	A	1536	1200	0	336	2000	1536
	M	1084	1200	-500	384	1500	1584
	J	2060	1200	-1200	2060	300	3260
	J	1640	1200	0	440	300	1640
	A	1117	1117	0	0	300	1117
	S	1190	440	750	0	1050	440
	O	1151	701	450	0	1500	701
	N	127	440	-313	0	1187	440
	D	1180	767	313	0	1500	767

	Month	Natural flow	Plant flow	Storage	Waste	Total storage	Total outflow
1927	J	5800	1200	500	4100	2000	5300
	F	7310	1200	0	6110	2000	7310
	M	8400	1200	0	7200	2000	8400
	A	1598	1200	0	398	2000	1598
	M	745	1200	-500	45	1500	1245
	J	950	1200	-1200	950	300	2150
	J	1242	1200	0	42	300	1242
	A	2296	1200	0	1096	300	2296
	S	1994	794	1200	0	1500	794
	O	1830	1200	500	130	2000	1330
	N	1580	1200	0	380	2000	1580
	D	1052	1200	-148	0	1852	1200
1928	J	883	1200	-317	0	1535	1200
	F	1172	1200	-28	0	1507	1200
	M	2180	1200	493	487	2000	1687
	A	876	1200	-324	0	1676	1200
	M	247	1200	-953	0	723	1200
	J	7060	1200	-423	6283	300	7483
	J	2038	1200	0	838	300	2038
	A	1960	1200	0	760	300	1960
	S	993	440	553	0	853	440
	O	1778	1131	647	0	1500	1131
	N	1860	1200	500	160	2000	1360
	D	1453	1200	0	253	2000	1453

	Month	Natural flow	Plant flow	Storage	Waste	Total storage	Total outflow
1929	J	706	1200	-494	0	1506	1200
	F	706	712	-6	0	1500	712
	M	1613	1200	413	0	1913	1200
	A	753	1200	-445	0	1468	1200
	M	106	874	-768	0	700	874
	J	268	668	-400	0	300	668
	J	4240	1200	0	3040	300	4240
	A	4200	1200	0	3000	300	4200
	S	675	440	235	0	535	440
	O	644	440	204	0	739	440
	N	463	440	23	0	762	440
	D	713	440	273	0	1035	440
1930	J	2740	1200	965	575	2000	1775
	F	2940	1200	0	1740	2000	2940
	M	1620	1200	0	420	2000	1620
	A	314	1200	-886	0	1114	1200
	M	109	523	-414	0	700	523
	J	307	707	-400	0	300	707
	J	700	700	0	0	300	700
	A	332	440	-108	0	192	440
	S	460	440	20	0	212	440
	O	414	440	-26	0	186	440
	N	381	440	-59	0	127	440
	D	339	440	-101	0	26	440

	Month	Natural flow	Plant flow	Storage	waste	Total storage	Total outflow
1931	J	735	440	295	0	321	440
	F	805	440	365	0	686	440
	M	1543	729	814	0	1500	729
	A	459	859	-400	o	1100	859
	M	275	675	-400	0	700	675
	J	304	704	-400	0	300	704
	J	2490	1200	0	1290	300	2490
	A	1340	1200	0	140	300	1340
	S	1100	440	660	0	960	440
	O	706	440	266	0	1226	440
	N	798	524	274	0	1500	524
	D	706	706	0	0	1500	706
1932	J	2510	1200	500	810	2000	2010
	F	3408	1200	0	2208	2000	3408
	M	2886	1200	0	1686	2000	2886
	A	886	1200	-314	0	1686	1200
	M	318	1200	-882	0	804	1200
	J	752	1200	-504	56	300	1256
	J	3766	1200	0	2566	300	3766
	A	2390	1200	0	1190	300	2390
	S	1808	608	1200	0	1500	608
	O	1757	1200	500	57	2000	1257
	N	1507	1200	0	307	2000	1507
	D	992	1200	-208	0	1792	1200

	Month	Natural flow	Plant flow	Storage	Waste	Total storage	Total outflow
1933	J	1592	1200	208	184	2000	1384
	F	2780	1200	0	1580	2000	2780
	M	5015	1200	0	3815	2000	5015
	A	1635	1200	0	435	2000	1635
	M	523	1200	-677	0	1323	1200
	J	3510	1200	-1023	3333	300	4533
	J	4040	1200	0	2840	300	4040
	A	3320	1200	0	2120	300	3320
	S	1695	495	1200	0	1500	495
	O	1585	1200	385	0	1885	1200
	N	894	1200	-306	0	1579	1200
	D	823	902	-79	0	1500	902
1934	J	1080	1080	0	0	1500	1080
	F	3020	1200	500	1320	2000	2520
	M	3180	1200	0	1980	2010	3180
	A	2120	1200	0	920	2000	2120
	M	735	1200	-500	35	1500	1235
	J	1433	1200	-1200	1433	300	2633
	J	6690	1200	0	5490	300	6690
	A	5520	1200	0	4320	300	5520
	S	5150	1200	1700	2250	2000	3450
	O	5130	1200	0	3930	2000	5130
	N	3550	1200	0	2350	2000	3550
	D	2430	1200	0	1230	2000	2430

Economic size of reinforced concrete penstock.

Assume:

Cost of reinforced concrete = \$10 per cu. yd.

Cost of steel = 4.5 ¢ per lb.

Fixed charges =12.5 %

Undeveloped water power = 0.3 ¢ per kw-hr.

hf = loss of head per ft. of penstock = $\frac{kv^2}{d^{1.25}\cdot 2g}$, where

$k = .029$, v = average velocity of water flowing through penstock, d = diameter of pipe, in ft.

Section of pipe, as shown in fig. 17 = $0.369d^2$

Efficiency of turbine = 0.88

Efficiency of generator = 0.94

f = tensile strength of steel = 16,000 lb. per sq. in.

Cost per ft. of pipe will be:

Cost of concrete = $\frac{10}{27} \times .369d^2$ = $0.1366d^2$ in dollars

Cost of reinforcing steel

assume head = 40 ft., tensile stress on steel = $\frac{pd}{2}$ = 1,250d

area of steel required, a = 1,250d ÷ 16,000 = 0.078d sq.in.

volume of steel per ft. of pipe = πda = $3.19d^2$ cu.in.

Cost of steel = $0.045 \times 0.384 \times 3.19d^2$ = \$$0.0408d^2$

Cost of temperature steel

assume area of temperature steel = .3% =

$= .003 \times .369d^2 \times 12^2$

$= .1593d^2$ sq. in.

Cost of steel = $0.045 \times 0.384 \times 12 \times 0.1593d^2$ = \$0.0245

Total cost per ft. of pipe $= 0.1366d^2 + 0.0408d^2 + 0.0245d^2$

$= 0.2019d^2$ in dollars

Annual fixed charges = Ci $= 0.2019d^2 \times 0.125 = 0.0252d^2$ in dollars

Value of power lost due to friction per ft. of pipe

assume $Q_{ave.} = 0.7Q_{max.} = 840$ c.f.s.

$h_f = \frac{0.29v^2}{d^{1.25} \times 2g} = \frac{515}{d^{5.25}}$ in ft. per ft. of pipe

power lost in kw. $= (Q_{ave.} \times h_f \times .88 \times .94 \times .746) \div 8.8$

$= \frac{30,300}{d^{5.25}}$

Value of friction loss per ft. of pipe = Pb $= .003 \times \frac{30,300}{d^{5.25}} \times 8,760$

$= \frac{796,000}{d^{5.25}}$ \$/year

Table

Annual Loss per ft. of Pipe, in Dollars

d	$d^{5.25}$	Pb	Ci	Pb+Ci
11	293,300	2.72	3.05	5.77
12	464,000	1.72	3.63	5.35
13	704,000	1.13	4.25	5.38
14	1,043,000	0.77	4.94	5.71

Economic size = 12 ft. diameter

Cost $= 3.63 \div 1.25 =$ \$29 per ft. of pipe

Dam sections.

Assumptions

w_1 = unit weight of concrete = 145 lb. per cu. ft.

w_2 = unit weight of water = 62.5 lb. per cu. ft.

w_3 = unit weight of silt = 62.5 lb. per cu. ft.

c = ratio of area subjected to uplift to the whole area = 0.5

f = coefficient of static friction = 0.75 for concrete on concrete.

H = height of max. flood level above spillway crest, in ft.

h = height of dam, in ft.

A = cross-sectional area of dam, in sq. ft.

l = length of joint, in ft.

Y = distance from the upstream extremity of the joint to some point of reference, in ft.

The dams are designed for max. water pressure with uplift, and also for max. water pressure with silt up to 10 ft. from spillway crest but without uplift.

Dam, Plant No. 2, spillway section

H = 25 ft.

h	A	l	Y	H + h
5	78	22.5	0	30
10	204	29.8	0	35
15	368	35.7	0	40
20	560	40.8	0	45
25	775	45.4	0	50
30	1012	49.5	0	55
40	1544	56.7	0	65
50	2144	63.2	0	75
60	2808	69.3	0	85
70	3539	74.8	0	95
80	4314	80.0	0	105
90	5130	87.9	0	115
100	6034	102.0	0	125

Dam, Plant No. 3, Spillway section, H = 20 ft.

5	70	19.5	0	25
10	186	26.5	0	30
15	332	31.7	0	35
20	502	36.2	0	40
30	906	44.2	0	50
40	1380	50.6	0	60
50	1915	57.6	0	70
60	2697	72.0	0	80

Dam, Abutment section, Top width = 9 ft, a = 5 ft.

h	A	l	Y	a + h
14.5	176	9.o	0	19.5
20.0	232	11.4	0	25.0
25.0	291	14.2	0	30.0
28.0	336	16.1	0	33.0
33.0	427	20.1	0.6	38.0
38.0	538	24.4	1.2	43.0
45.0	731	30.6	2.0	50.0
52.0	967	36.9	2.8	57.0
60.0	1284	41.1	3.1	65.0
70.0	1740	50.5	3.4	75.0
80.0	2295	60.0	3.9	85.0

Design and Cost Estimates, Plant No. 2.

Intake and crest gates

max. water level, elev. =1328

crest of spillway, elev. =1303

height of crest gates = 20ft.

Rocks

$Q_{aver.} = .70\ Q_{max.} = 840$ c.f.s.

assume velocity through gross area of rocks = 1.75 ft./sec.

area required = 840 ÷ 1.75 = 480 sq. ft., use 2-10' x 24' racks

center of penstock 6' below spillway crest level or 24'

below normal w.l.

Gates

assume velocity through gates $= .075\sqrt{2gh}$

head normal water level = 1321

normal tailwater level at plant = 1158

friction loss in intake and waterway = 10 assumed effective

head = 153'

$V = .075\sqrt{2g \times 153} = 8$ sec. ft.

area required = 1,200 ÷ 8 = 150 sq. ft.

use 2 - stoney gates with 7' x 12' openings

Capacity of crest gate traveling hoist

Head to center line of gate = 15'

Weight of taintor gate 20 x 20 = 40,000lb. assumed

Creager table 37, page 317

Weight of stoney gate 20 x 20 = 6.4 x 20 x 20 x 15

= 38,400 lb. (Creager p.313)

The 40,000 lb. taintor gates is assumed.

As the force required to lift a taintor is about equal to the wt. of gate,

Hoist capacit y = 25 tons.

for safety 2 - 25 tons. hoists will be used.

Capacity of intake gates hoist

Head on c.l. of gate = 31 ft.

wt. of 7' x 12' stoney gates = 4.1 x 31 x 7 x 12

= 10,700 lbs. (Creager, P.313)

Capacity of hoist = 8 tons.

Surge tank (see Creager p.530)

Length of 12' concrete pipe = 340 ft.

Length of concrete lined tunnel = 4,000 ft.

Length of 12' steel pipe = 320 ft.

Length of 8' branch pipes = 300 ft. each

Acceleration

1. assume $q_2 = 1200$

$q_1 = 800$

V_2 = 12 on tunnel 10.6 on penstock

V_1 = 8 on tunnel 7.0 on penstock

V/C_a = 12.0/100 = 0.12 for tunnel (R = 2.75')

10.6/100 = 0.106 for concrete pipe [illegible]

$$f_a = 3.6 \times 4000 + 2.7 \times .340 + 1.5 \times .081 + \frac{10.5^2}{2g}$$

$$= 17.4$$

C_r = 100 (assumed)

$$f_r = 2.1 \times 4000 + 1.4 \times .34 + 1.2 \times .081 + \frac{10.5^2}{2g}$$

$$= 10.7$$

2. $c_a = f_a/V_2^2 = 17.4/\ 144 = 0.121$

$\Sigma Al = 4000 \times 100 + 420 \times 113 = 400{,}000 + 47{,}500$

$= 447{,}500$

$$F = \left(\frac{N_a'}{100\ C\ V_2\sqrt{\frac{2g}{AL}}} \right)^2 = N_a'^2/3.02$$

3. $V_1 = 8.0$

4. $K_a' = \dfrac{100\ C\ (V_2^2 - V_1^2)}{y_a} = 968/y_a$

5. Assume y_a =	10	12	15	20	25
6. K_a' =	96.8	80.7	64.5	48.4	38.7

7. $P = \dfrac{V_2 - V_1}{V_2} = .333$

8. Curve no. 1

(page 29) N_a'	76.5	58.0	44.3	32.0	24.2
9. F =	1940	1110	650	340	194

10. assume $d_a = 10$

departure =

$.121 \times 8^2 + 10$	17.7	17.7	17.7	17.7	17.7
11. Lowest surge	27.7	29.7	32.7	37.7	42.7

12. dia. of riser pipe =12'

R = 113

D =	51.2	39.6	31.2	24.0	19.7

Retardation

1. $c_r = 10.7/V_2^2 = .0765$
2. $y'_r = 100\ c\ V_1^2/\ K'_r = 1070/K'_r$
3. $N'_r = 100\ c\ V_2\sqrt{(2g/A1)}\ \sqrt{F} = 1.10\sqrt{F}$

4. F =	1940	1110	650	340	194
5. N'_r =	48.5	36.7	28.1	20.3	15.3
6. K'_r (curve no.4) P. 529	42.6	33.2	26.1	20.0	15.3
7. Y_r	25.0	32.2	41.0	53.5	70.8
8. $d_r = 5$ departure = 5 - 8^2 x .0765)	.1	0.1	0.1	0.1	0.1
9. highest surge	25.1	32.3	41.1	53.6	70.9

Test for incipient stability

R = 113 sq. ft. H = 149, C = .0765

$$D_m = \sqrt{\frac{R + \frac{AL}{2gHc}}{0.785}} = 30.2$$

Diameter used = 32'

Lowest surge = 40'

Highest surge = 32'

A tank of 75' high will be used with bottom at elevation 1281 or 42' below crest of taintor gate.

Thickness of plate

total pressure = 75 x 62.5 x 32/2 = 75,000 lbs. per ft.

assume f_s = 12,000 lb./ sq in.

Joint eff. = .70

thickness of plate = 75000/(12 x 12000 x .7) = .742"

use 3/4 inch for lowest 25'

use 1/2 in. for next 25'

1/4 in. for next 25'

Characteristics of turbines

Head water level = 1321, 2' below crest of taintor gate

Tail water level = 1158

Loss through intakes and water-way = 10 ft. (assumed)

Effective head = 153 ft.

Assume three units are used Q = 400 c.f.s. each

N_s = 5050/(H + 32) + 19 = 46.3

h.p. = 400 x 153 x .95 /8.8 = 6600

R.P.M. = $N_s H^{\frac{5}{4}}/\sqrt{hp}$ = 308

assume R.P.M. = 300 which is a synchronous speed

N_s = R.P.M. x $\sqrt{hp}/H^{\frac{5}{4}}$ = 45.2

The turbine used is 6600 hp. ,300 r.p.m. N_s = 45.2

Discharge dia. = 56"

Inlet diam. = 56 x .96 = 54" (Creager p. 603-604)

generator capacity = 6600 x .746 x .94 = 4600

=5750 kva. at 80 % p.f.

Distance c.l. to c.l. of units = 4.2 D approx. (Creager p.609)

= 19.6 ft.,

22 ft will be assumed.

Cost estimation.

Dam.

cross-section of river is shown in fig. 17

height of dam = 100 ft.

net length of spillway = 240 ft. with 12 taintor gates

volume of dam including abutaments, sill, piers etc.

= 100,000 cu. yd.@\$7 = \$700,000

excavation, rock and earth,,56,000 cu. yd.

@\$.3== \$ 16,800

cofferdam		\$ 20,000
12 20x20'taintor gates 480,000 lb.	@\$.1 =	\$ 48,000
2 20-ton traveling hoists	@ \$3,000 =	\$ 6,000

Total cost of dam

concrete	\$700,000
excavation	16,800
cofferdam	20,000
taintor gates and hoists	54,000
land and property (assumed value)	30,000
miscellaneous cost	19,200
	\$850,000

Intake and waterway

racks, area = 2x10x24 = 480 sq. ft. @\$ 2.5	=	\$ 1,200
2 Stoney gates, wt. = 21,400 lb. \$.090	=	2,000
superstructure 18x24x30 = 13,000 cu. yd.@\$.15	=	1,950
substructure, 1,000 cu.yd.@\$8	=	8,000

excavation, rock and earth, 1,500 cu. yd.	@$.5	=$	750
vent pipe, 3 ft. diameter, 50 ft. long	@ $12	=	600

total cost of intake $14,500

reinforced concrete pipe, 12' dia., 340 ft.	@$36	=	12,300
concrete lined tunnel, 100 sq. ft., 4,000'	@ $70	=	280,000
steel pipe, 12 ft. diameter, 320 ft.	@ $56	=	17,900
steel pipe, 8 ft. diameter, 900 ft.	@ $40	=	36,000

total cost of waterway (including excavation, anchorages) =$346,200

surge tank, 75 ft. high, 32 ft. diameter, average thickness = 1/2 in., total weight of steel plates is equal to 154,000 lb., add 30% for rivets, stiffeners etc. total wt. is about 200,000 lb. @$.09 =$ 18,000

penstock valve	$ 10,000
excavation and foundation	$ 8,000
appurtenances	$ 3,000
total cost of surge tank	$ 39,000
Total cost of intake and waterway	$400,000

superstructure 46x50x88 = 202,000 cu. ft. @$.3 =	$ 60,600
substructure, 10,300 cu. yd. of concrete @$ 9 =	$ 92,700
foundation, excavation, etc	$ 4,700
Total cost of superstructure and substructure	$158,000

Total cost of equipment 19,800 hp. @$26= $515,000

Plant No. 3. Designs and cost estimation

Designs.

Intake.

Racks

Q = 1200 c.f.s.

3 units, Q = 400 c.f.s.

Normal water elev. = 1,129.0

Max. flood elev. = 1,146.0

Crest of spillway elev. = 1,126.0

Crest of flashboards elev. = 1131.0

Normal Tail water elev. = 1066.0

H = 63.0ft.

assume velocity through gross area of rack = 1.75 ft./ sec.

$Q_{ave.}$ = 400 x .7 = 280

Area required = 280 ÷ 1.75 = 160 sq. ft.

Use 12 x 13.5 racks, area provided = 162 sq. ft.

Gates

Assume $V = .075\sqrt{2gh}$ = 4.76 ft. per sec.

Area required = 280 ÷ 4.76 = 58.7

Use 8' x 10 Broome gates, area provided = 80 sq. ft.

Turbines

$$N_s = \frac{5050}{H + 32} + 19 = \frac{5050}{63 + 32} + 19 = 73.1$$

e = 93 %

$$h_p = \frac{63 \times 400 \times .93}{8.8} = 2660$$

Total capacity of 3 units will be 7980 hp.

R.P.M. = $N_s\ H^{\frac{5}{4}} / h_p^{\frac{1}{2}}$ = 73.1 x $63^{\frac{5}{4}}$ / $\sqrt{2660}$ = 252.0

a generator of 252 R.P.M. at 60 cycles the no. of poles is 120 x 60 / 252 = 28.2

The next larger even number is 30 and R.P.M. is

120 x 60 / 30 = 240

The specific speed of turbine, N_s = R.P.M. x $\sqrt{h_p}$ / $H^{\frac{5}{4}}$

= 240 x $2660^{\frac{1}{2}}$ / $63^{\frac{5}{4}}$

= 69.7

Runner discharge diameter (Creager Fig.394, p.603)

= 63"

Distance center line to center line of unit = 4.2D

(Creager ,p.609) = 22ft.

23 ft. will be assumed.

Generator

60 f, 240 R.P.M.

assume eff. of generator = .94

capacity of generator = 2660 x .746 x .94 = 1870 kw.

or 2340 kva. at 80 p.f.

2250 kva. units will be assumed and the total capacity is 6750 kva.

Cost Estimation, Plant No. 3.

I. Dam

Concrete

abutments including power house substructure

17100 cu. y d. at \$7 = 119,700

spillway 350 long

33800 cu.yd. at \$7 = 236,600

foundation, sills and abutements

36000 cu. yd. at \$7 = 252,000

\$608,300

excavation 85,000 cu. yd. rock and earth a \$.3

= 25,500

cofferdam	20,000
flashboard 1750 sq. ft. @ 2.5=	4,400
land and property	20,000
miscellaneous	16,800
	\$ 695,000

II. Power house superstructure

173,000 cu. ft. @ \$.25	\$43,400
III. Equipment @ \$32 per hp. (7980 hp.)	\$255,600
Total cost	\$994,000

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GENERAL MAP—YUNG TING HO

Fig. 1

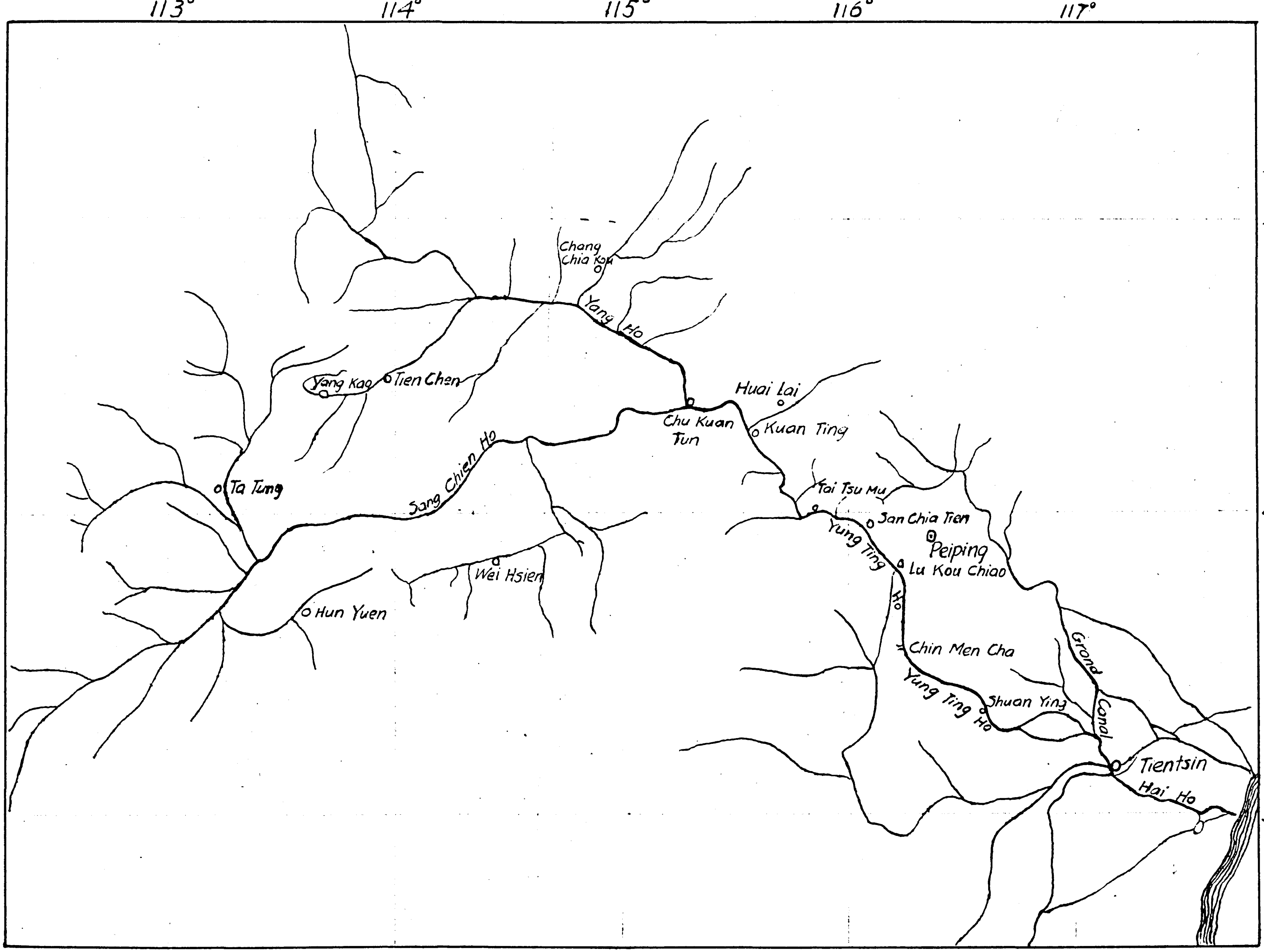
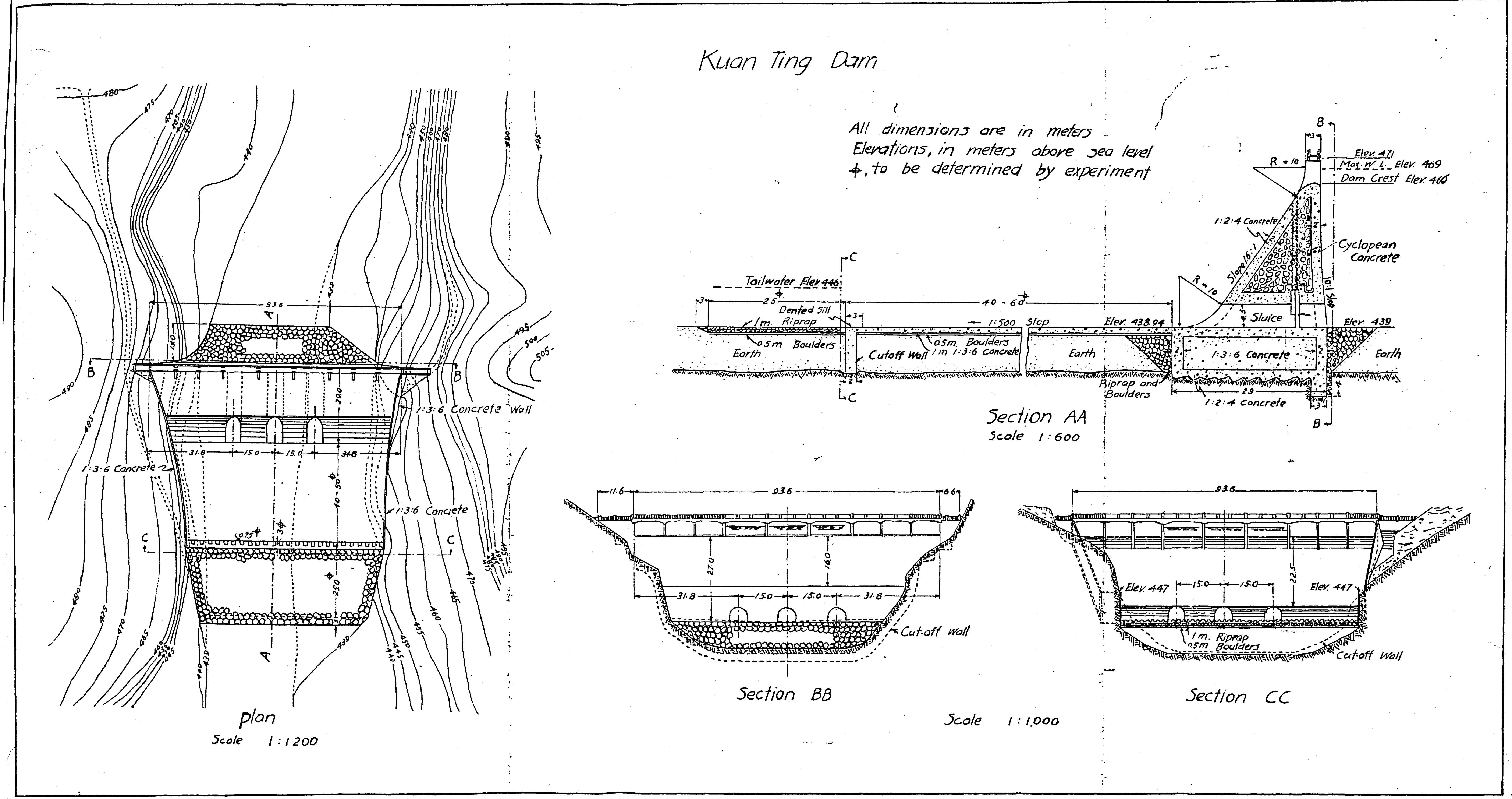


Fig. 2


Kuan Ting Dam
All dimensions are in meters
Elevations, in meters above sea level
⌖, to be determined by experiment
Plan
Scale 1:1200
1:3:6 Concrete Wall
1:3:6 Concrete
1:3:6 Concrete
Section AA
Scale 1:600
Tailwater Elev. 446
Dented Sill
1m. Riprap
0.5m Boulders
Earth
Cutoff Wall
0.5m. Boulders
1 m 1:3:6 Concrete
1:500 Slop
Elev. 438.94
Riprap and Boulders
1:3:6 Concrete
1:2:4 Concrete
Sluice
Elev. 439
Cyclopean Concrete
1:2:4 Concrete
Slope 1/6:1
R = 10
Elev. 471
Max. W. L. Elev. 469
Dam Crest Elev. 466
Section BB
Cut-off Wall
Section CC
Elev. 447
Elev. 447
1m. Riprap
0.5m Boulders
Cut-off Wall
Scale 1:1,000

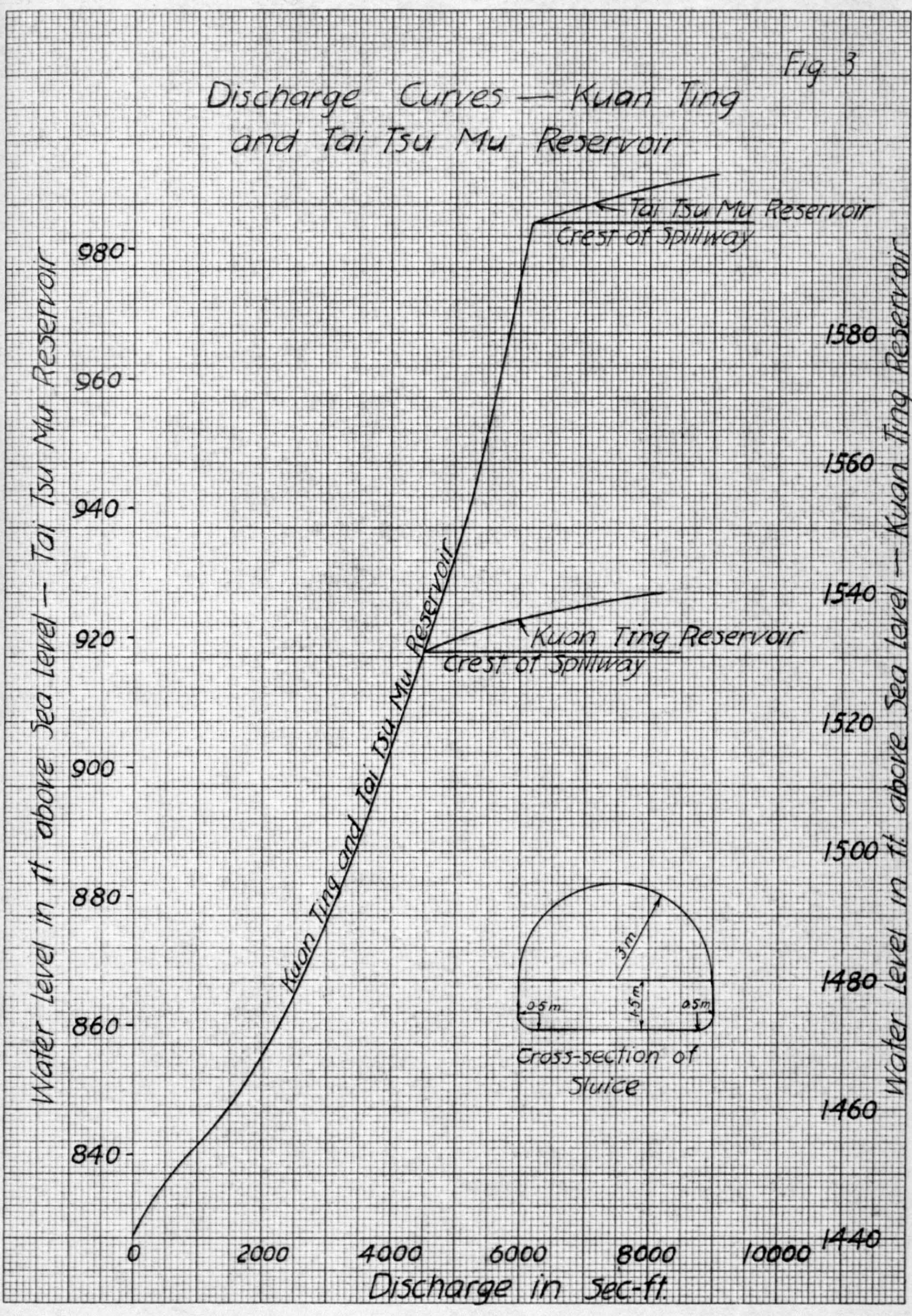
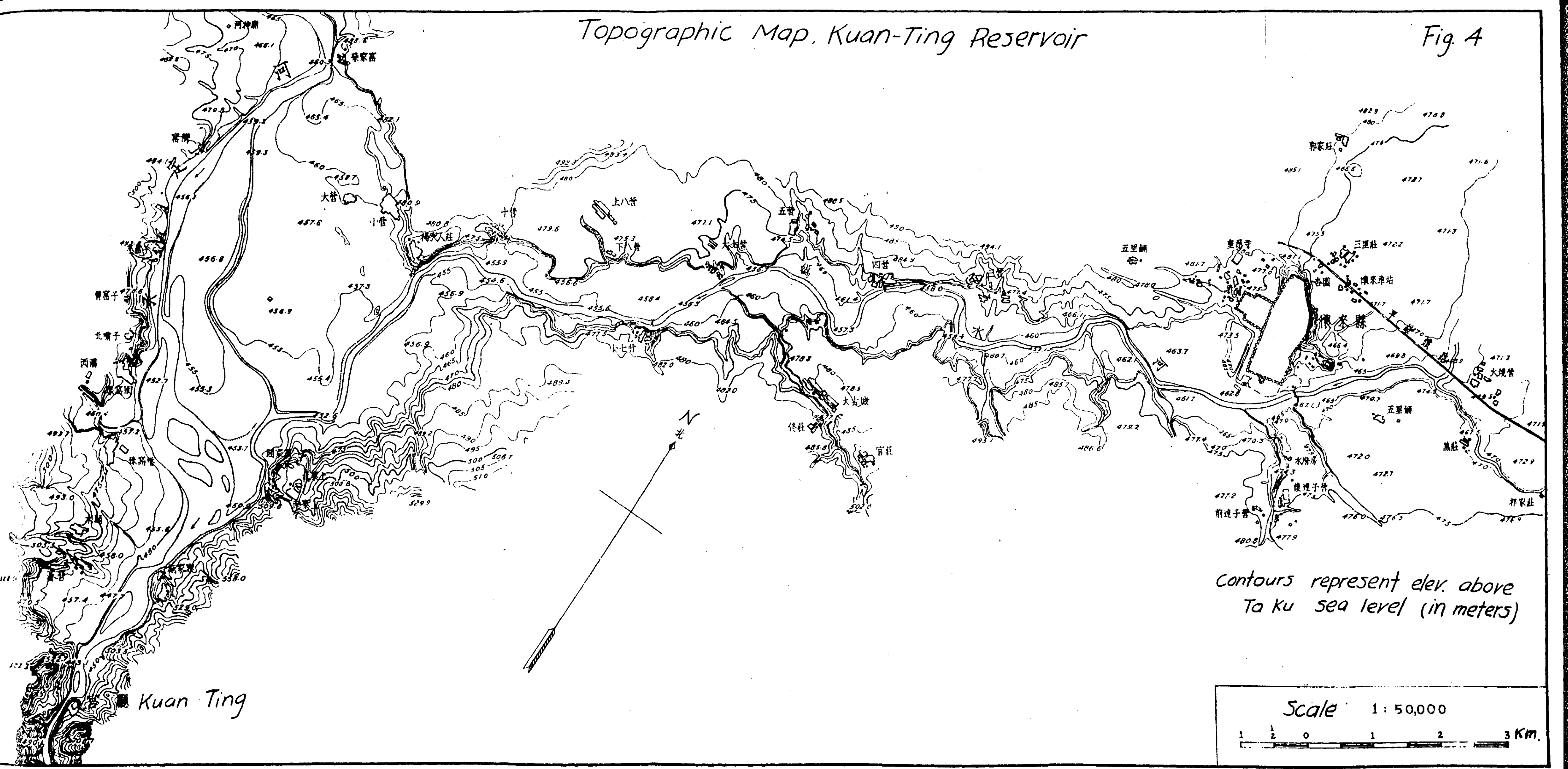


Fig. 3
Discharge Curves — Kuan Ting and Tai Tsu Mu Reservoir
Tai Tsu Mu Reservoir
Crest of Spillway
Kuan Ting and Tai Tsu Mu Reservoir
Kuan Ting Reservoir
Crest of Spillway
3m
0.5m
1.5m
0.5m
Cross-section of Sluice
Water Level in ft. above Sea Level — Tai Tsu Mu Reservoir
980
960
940
920
900
880
860
840
Water Level in ft. above Sea Level — Kuan Ting Reservoir
1580
1560
1540
1520
1500
1480
1460
1440
0
2000
4000
6000
8000
10000
Discharge in Sec-ft.



Topographic Map, Kuan-Ting Reservoir

Fig. 4

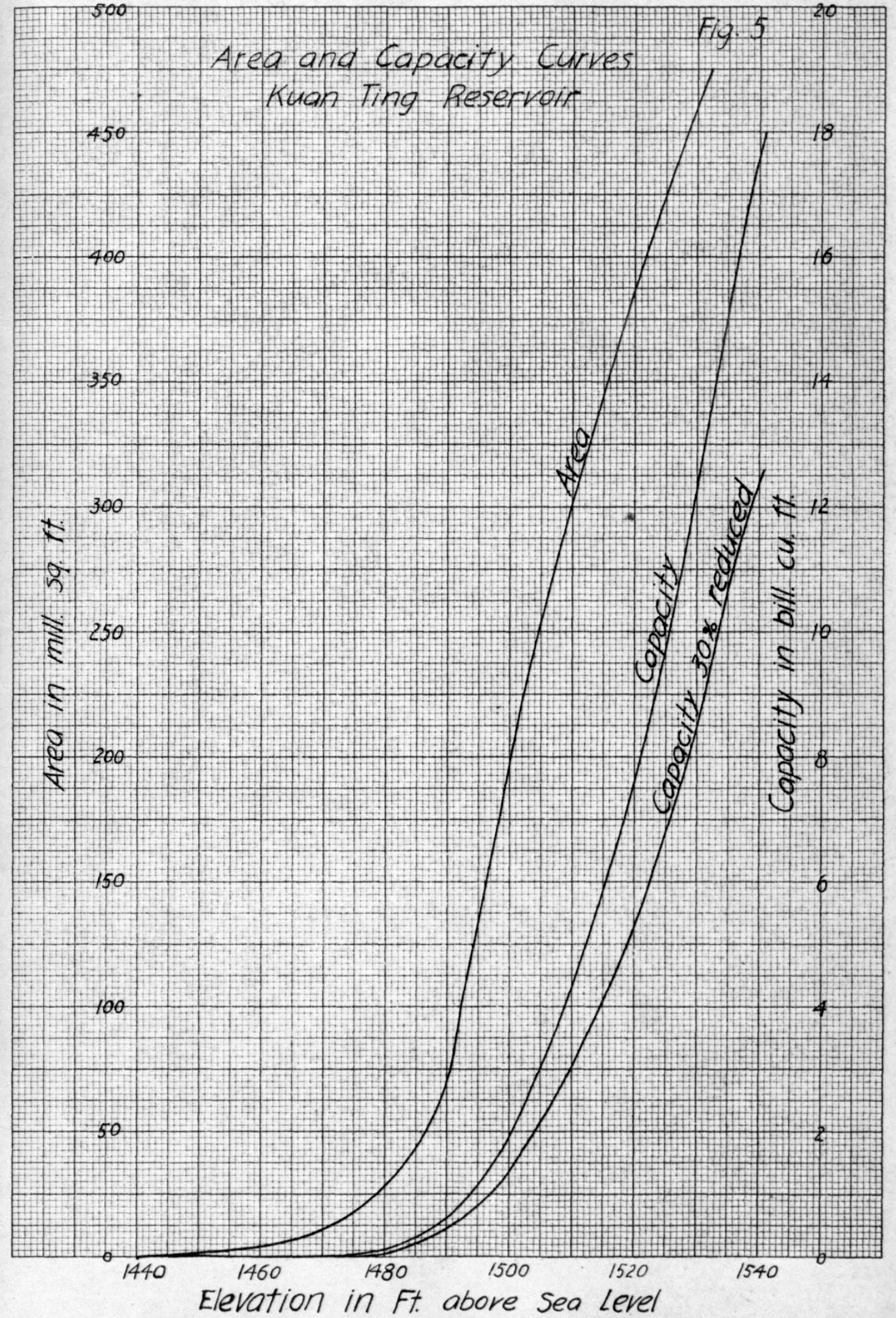


Fig. 5
Area and Capacity Curves
Kuan Ting Reservoir
Area
Capacity
Capacity 30% reduced
Area in mill. sq. ft.
Capacity in bill. cu. ft.
500
450
400
350
300
250
200
150
100
50
0
20
18
16
14
12
10
8
6
4
2
0
1440
1460
1480
1500
1520
1540
Elevation in Ft. above Sea Level

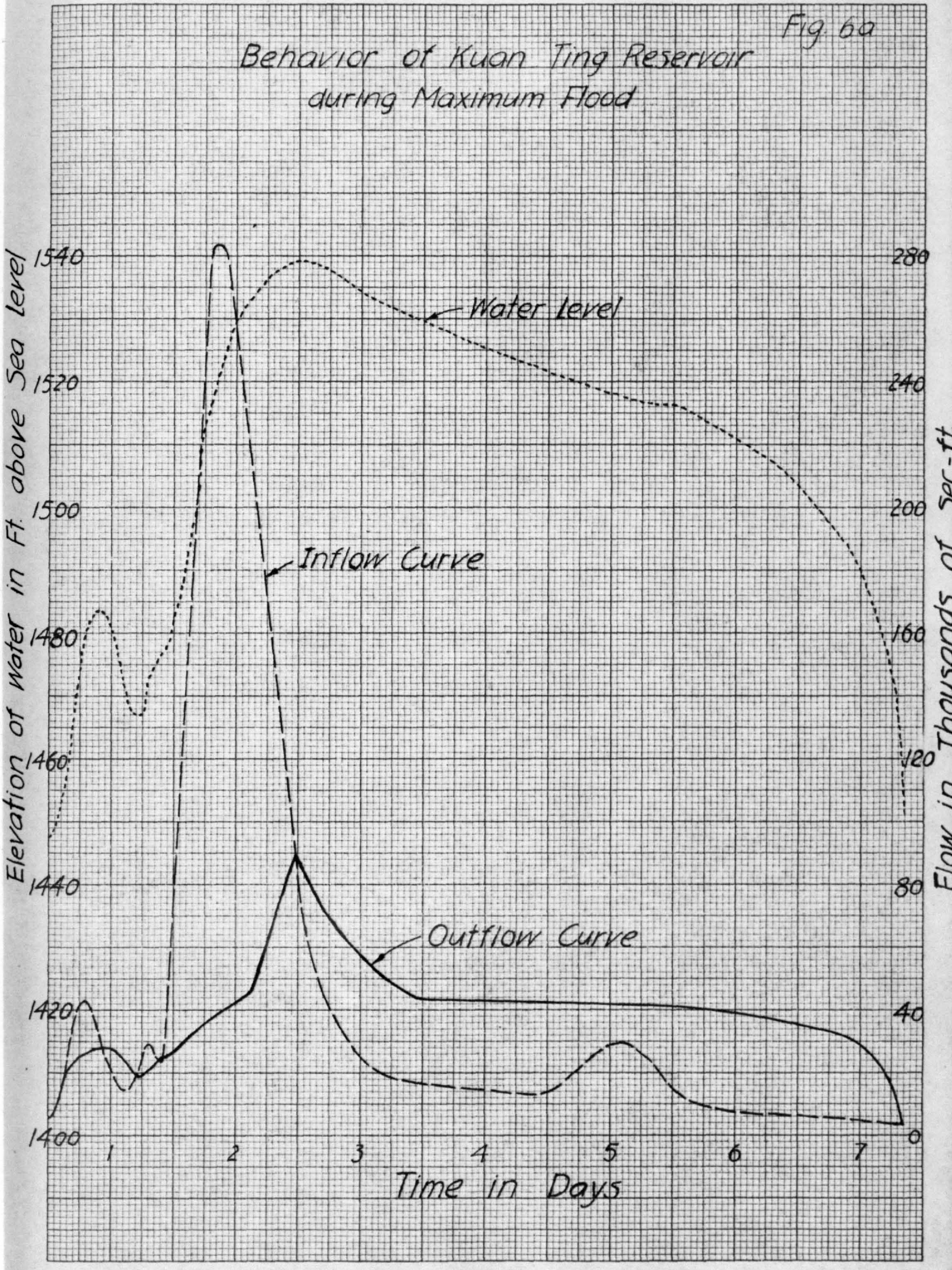
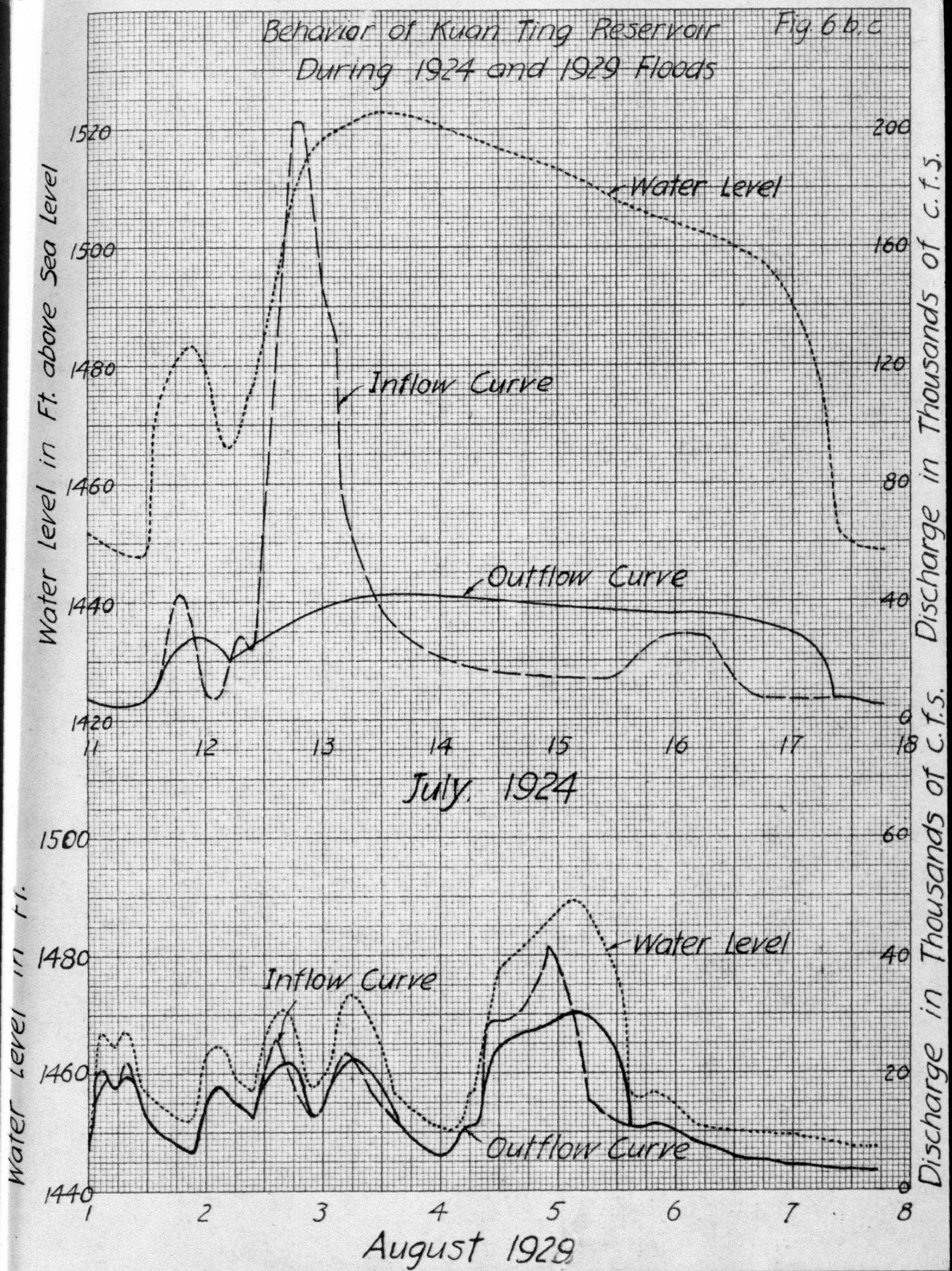


Fig. 6a
Behavior of Kuan Ting Reservoir
during Maximum Flood
Elevation of Water in Ft. above Sea level
Flow in Thousands of Sec-ft.
1540
1520
1500
1480
1460
1440
1420
1400
280
240
200
160
120
80
40
0
Water level
Inflow Curve
Outflow Curve
1
2
3
4
5
6
7
Time in Days


Behavior of Kuan Ting Reservoir
During 1924 and 1929 Floods
Fig 6 b, c
Water Level
Inflow Curve
Outflow Curve
Water Level in Ft. above Sea Level
Discharge in Thousands of c.f.s.
1520
1500
1480
1460
1440
1420
200
160
120
80
40
0
11
12
13
14
15
16
17
18
July, 1924
Water Level
Inflow Curve
Outflow Curve
Water Level in Ft.
Discharge in Thousands of c.f.s.
1500
1480
1460
1440
60
40
20
0
1
2
3
4
5
6
7
8
August 1929

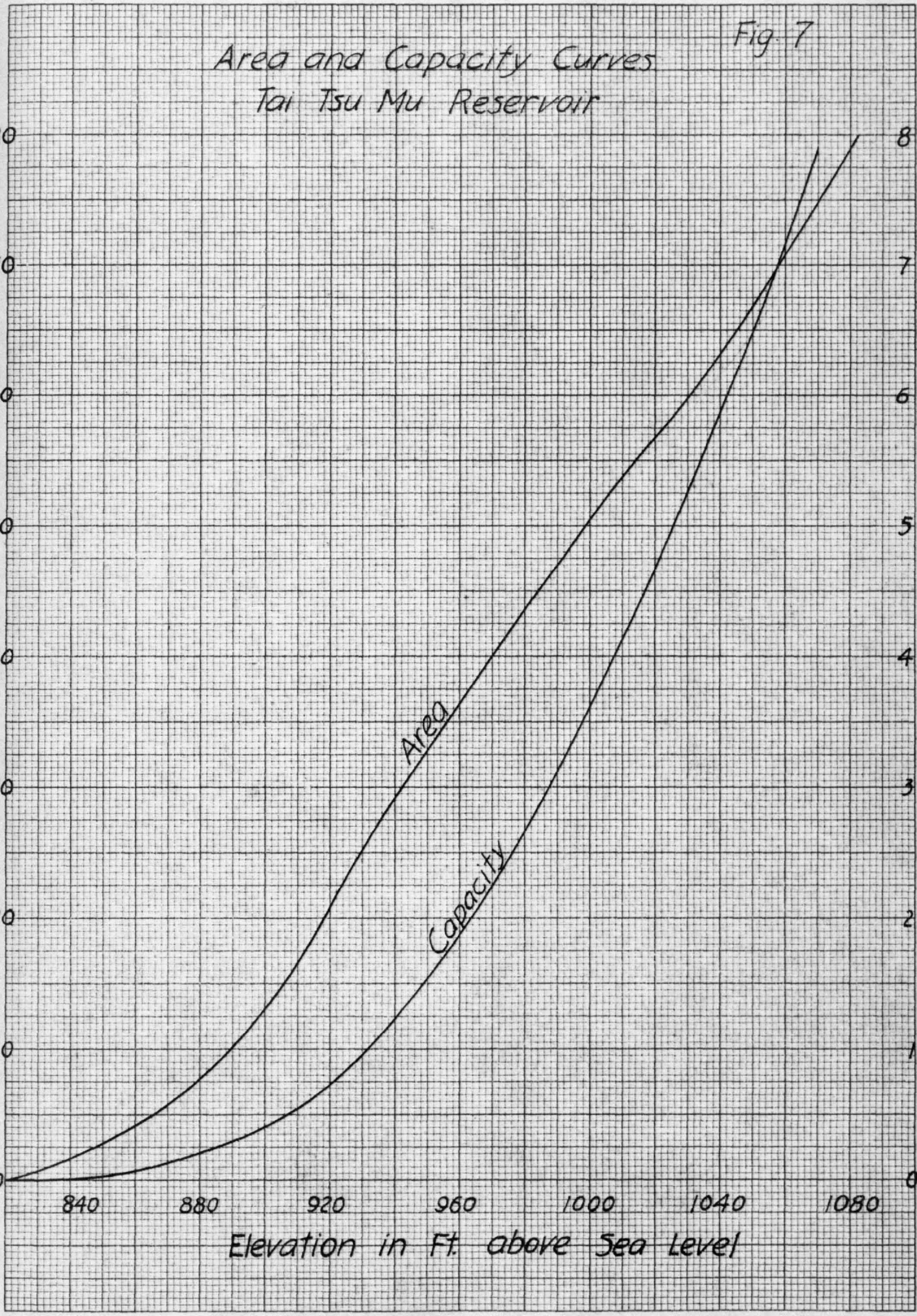


Fig. 7
Area and Capacity Curves
Tai Tsu Mu Reservoir
Area in mill. sq. ft.
Capacity in bill. cu. ft.
Elevation in Ft. above Sea Level
Area
Capacity
0
10
20
30
40
50
60
70
80
1
2
3
4
5
6
7
8
840
880
920
960
1000
1040
1080

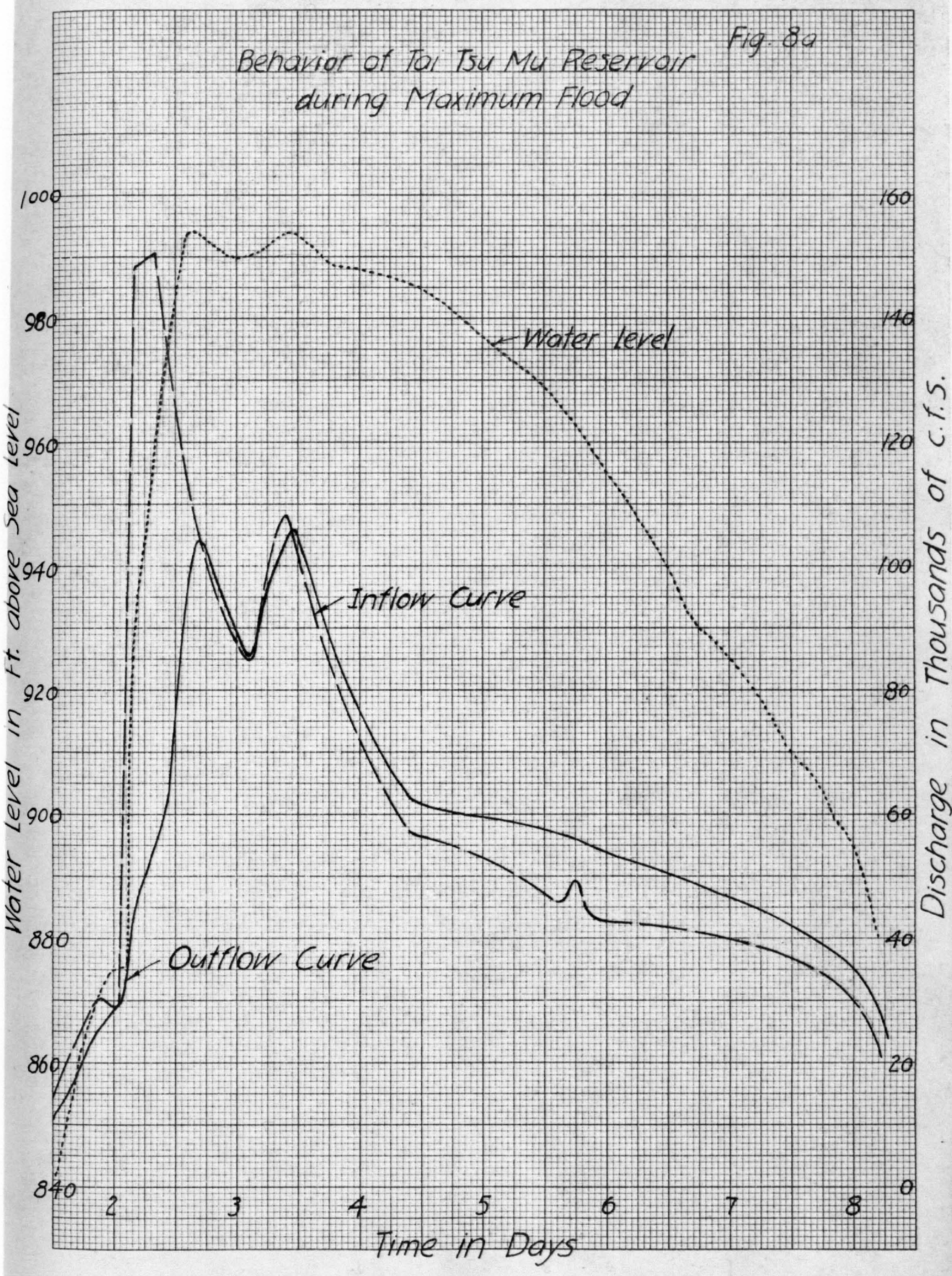
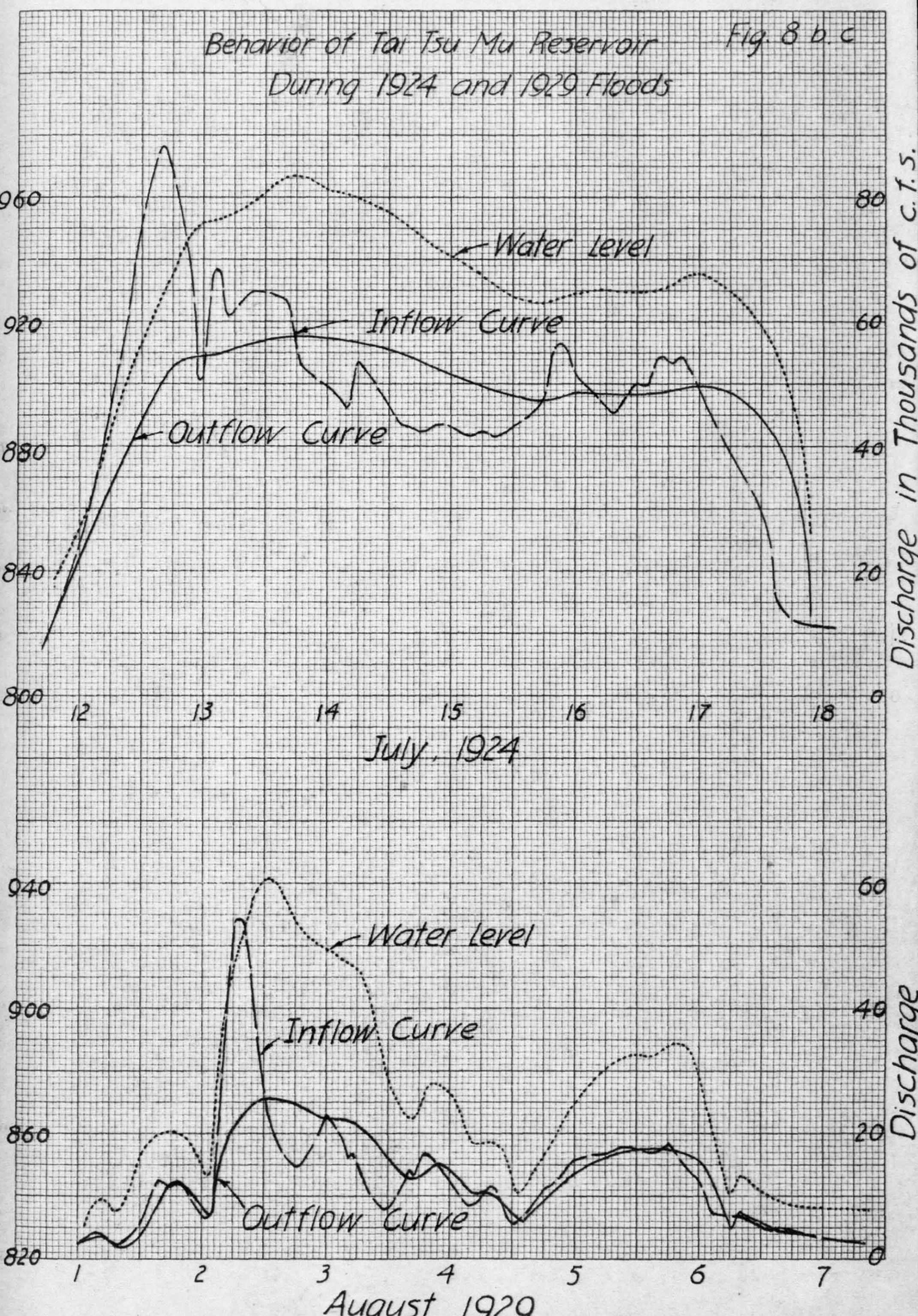


Fig. 8a
Behavior of Tai Tsu Mu Reservoir
during Maximum Flood
Water level
Inflow Curve
Outflow Curve
Water Level in Ft. above Sea Level
Discharge in Thousands of c.f.s.
Time in Days
1000
980
960
940
920
900
880
860
840
160
140
120
100
80
60
40
20
0
2
3
4
5
6
7
8



Behavior of Tai Tsu Mu Reservoir
During 1924 and 1929 Floods
Fig. 8 b. c
Water level
Inflow Curve
Outflow Curve
Elevation in Ft. above Sea Level
Discharge in Thousands of c.f.s.
960
920
880
840
800
80
60
40
20
0
12
13
14
15
16
17
18
July, 1924
Water level
Inflow Curve
Outflow Curve
Elevation
Discharge
940
900
860
820
60
40
20
0
1
2
3
4
5
6
7
August, 1929

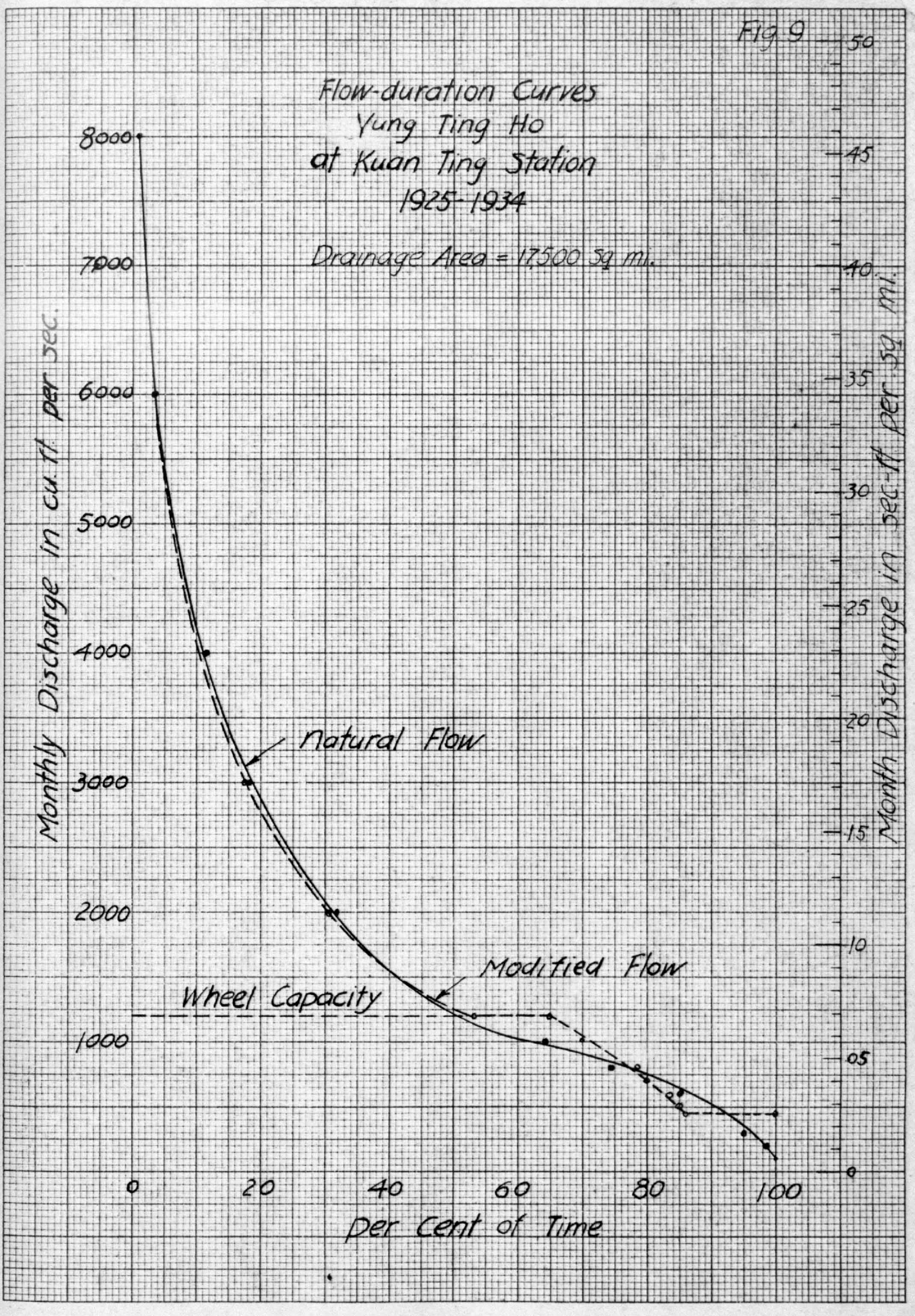


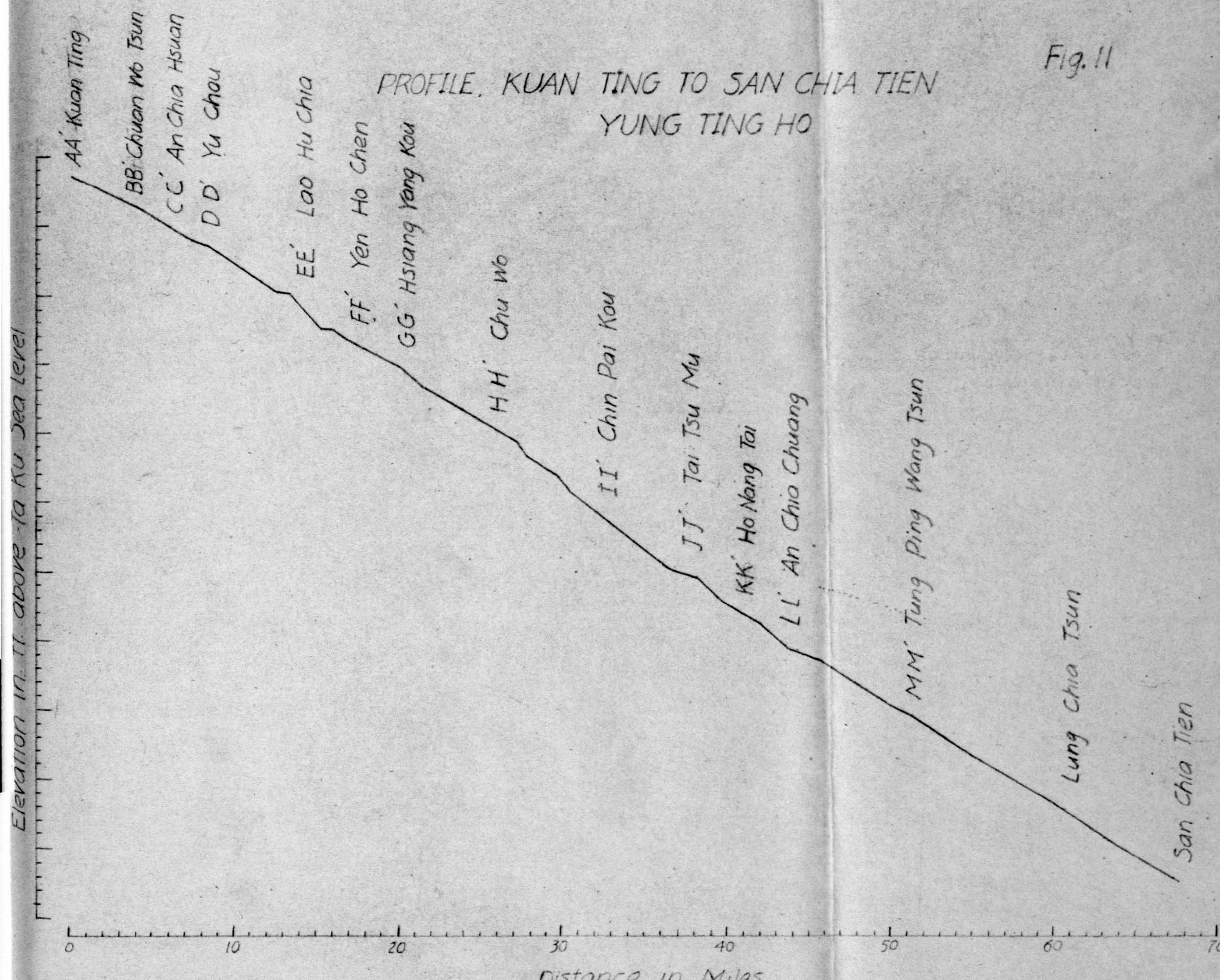
Fig 9
Flow-duration Curves
Yung Ting Ho
at Kuan Ting Station
1925-1934
Drainage Area = 17,500 sq mi.
Monthly Discharge in cu. ft. per sec.
Month Discharge in sec-ft per sq mi.
8000
7000
6000
5000
4000
3000
2000
1000
50
45
40
35
30
25
20
15
10
05
0
Natural Flow
Modified Flow
Wheel Capacity
0
20
40
60
80
100
Per Cent of Time

Fig. 10


TOPOGRAPHIC MAP — KUAN TO SAN CHIA TIEN
YUNG TING HO
Contours give the elevation in meters above Ta Ku sea level
官廳 Kuan Ting
安家漩 An Chia Hsuan
Plant No. 1
莊窩村 Chuang Wo Tsun
幽州 Yu Chou
Plant No. 1a
Plant No. 2
老虎岬 Lao Hu Chia
向陽口 Hsiang Yang Kou
沿河城 Yen Ho Cheng
沿河口
Plant No. 3a
Plant No. 3
東窩 Chu Wo
Chin Pei Kou
太子墓 Tai Tsu Mu
下馬嶺
Ho Nan Tai
安家莊 An Chia Chuang
Plant No. 4
Tung Wang Pin
西王平村
清水澗村
花披根
永 定 河
Plant no. 5 and plant no. 6 are located on Lung Chia Tsun and San Chia Tien respectively not shown in this fig.
N
Scale 1 : 50,000
Km.

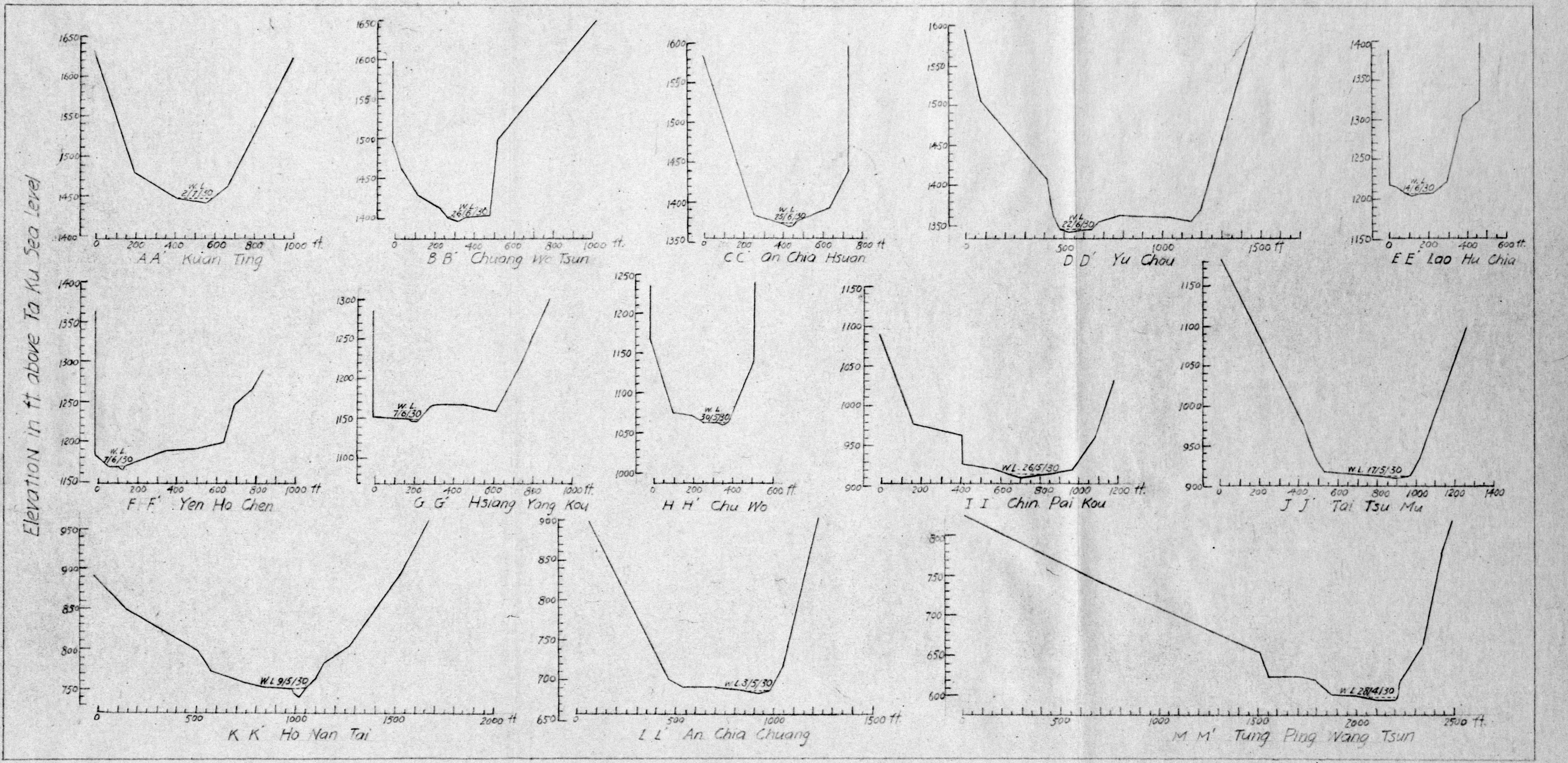
Fig. 11

PROFILE, KUAN TING TO SAN CHIA TIEN
YUNG TING HO



CROSS-SECTIONS KUAN TING TO SAN CHIA TIEN

Fig. 12



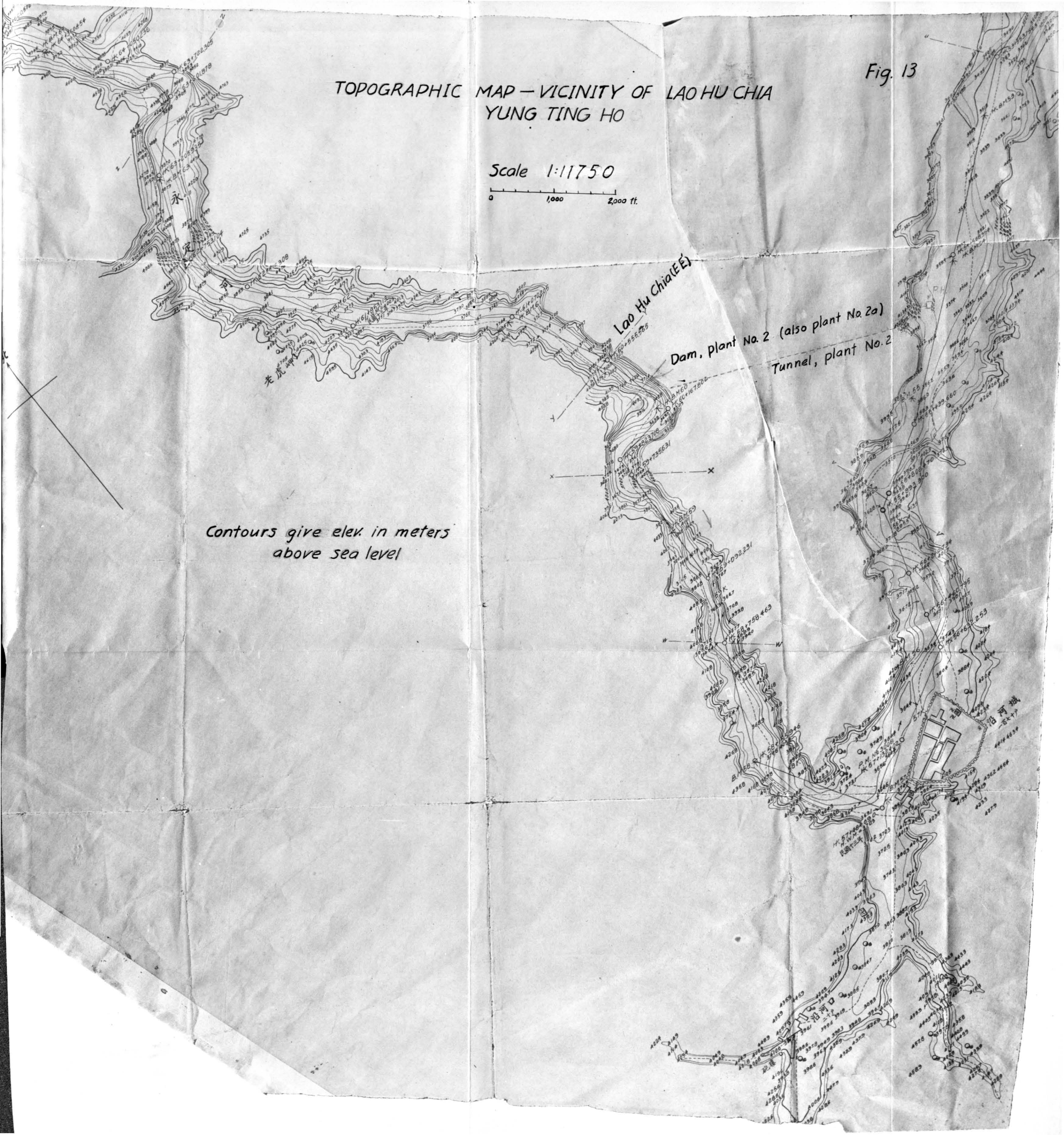


Fig. 13
TOPOGRAPHIC MAP — VICINITY OF LAO HU CHIA
YUNG TING HO
Scale 1:11750
0
1,000
2,000 ft.
Lao Hu Chia (E.E')
Dam, plant No. 2 (also plant No. 2a)
Tunnel, plant No. 2
Contours give elev. in meters
above sea level
永
定
河

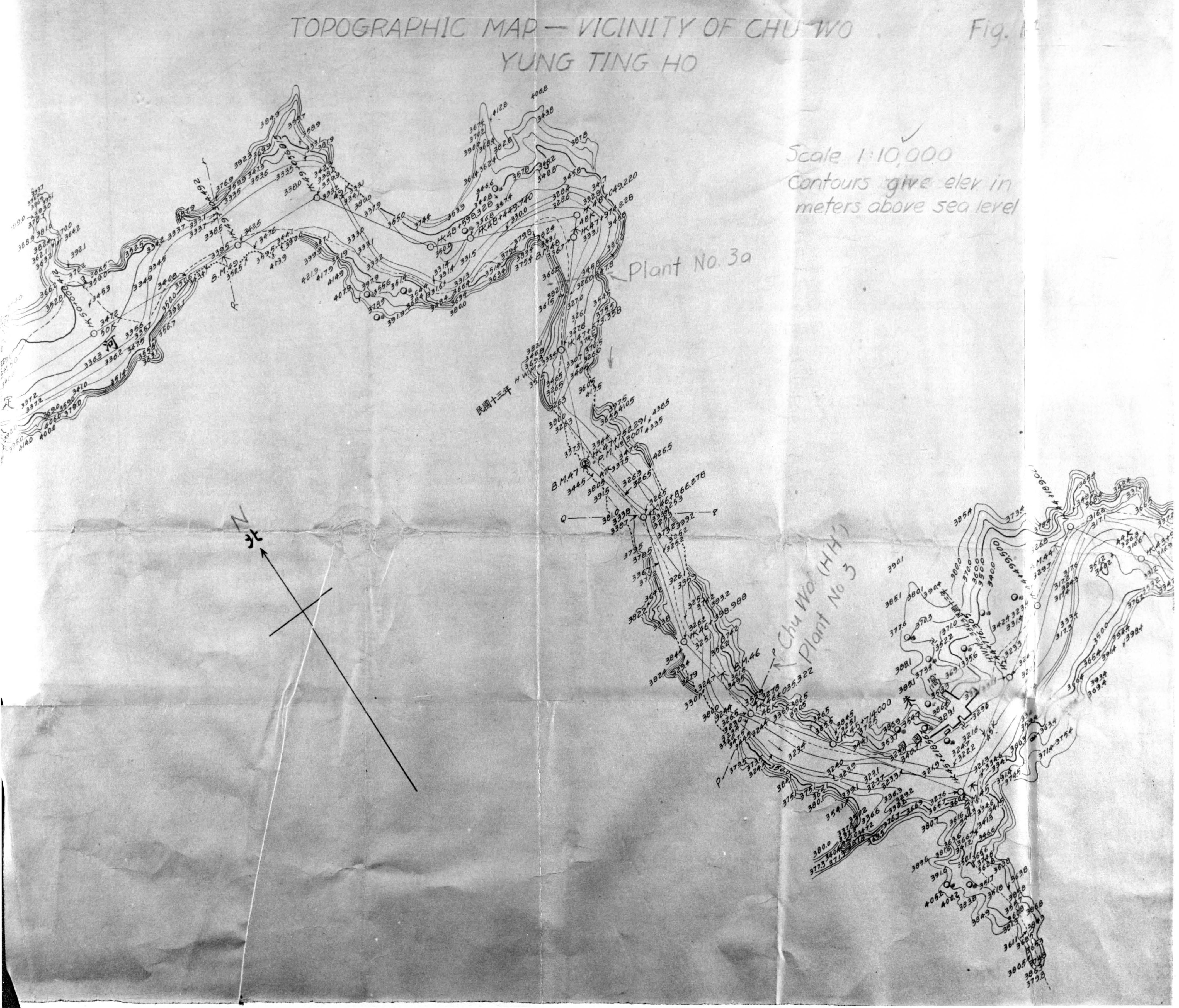
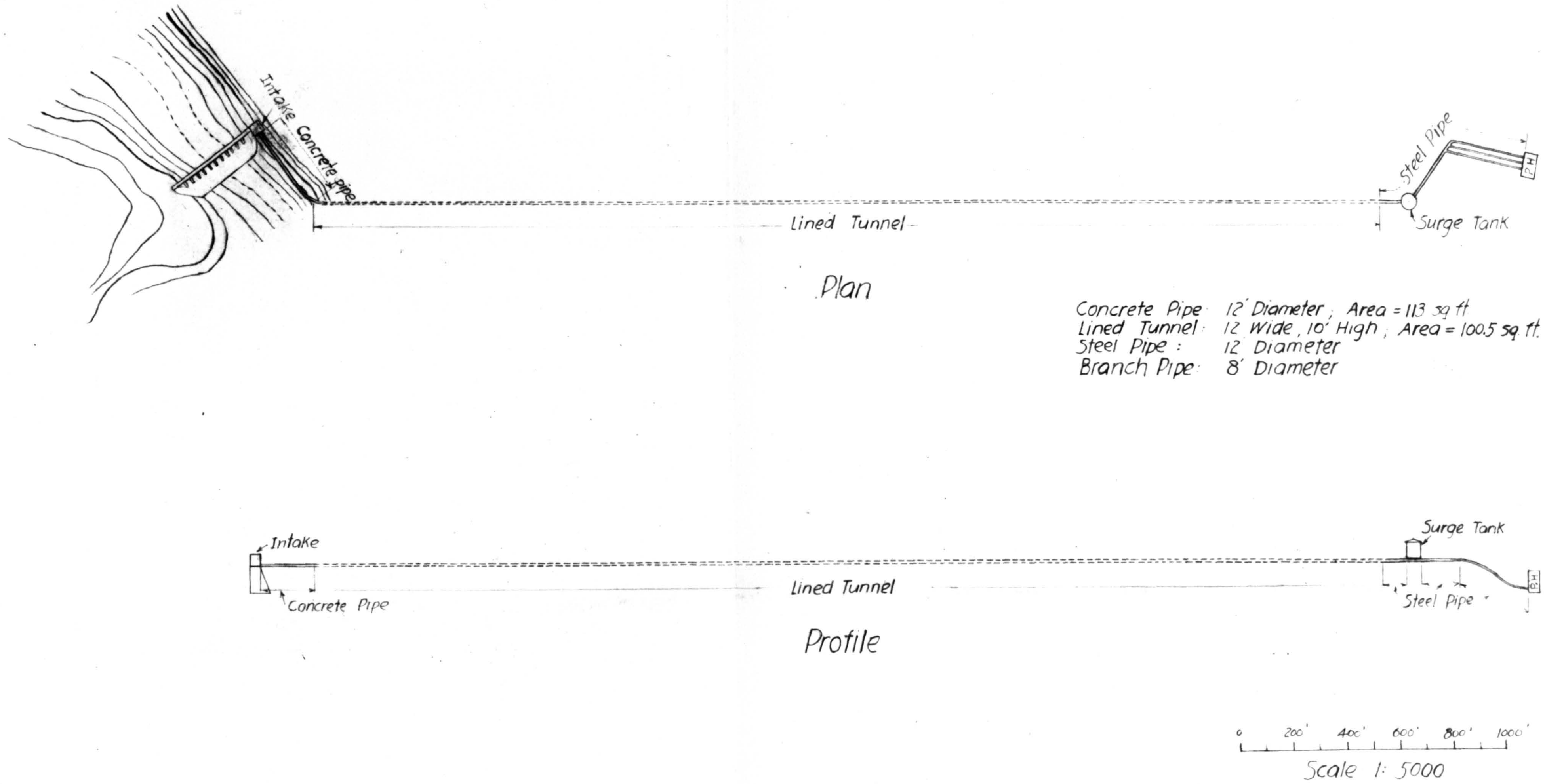

TOPOGRAPHIC MAP — VICINITY OF CHU WO
YUNG TING HO
Fig. 1
Scale 1:10,000
Contours give elev. in meters above sea level
Plant No. 3a
Chu Wo (H.H.') Plant No. 3
N
北
B.M. 47
B.M. 46
B.M. 49

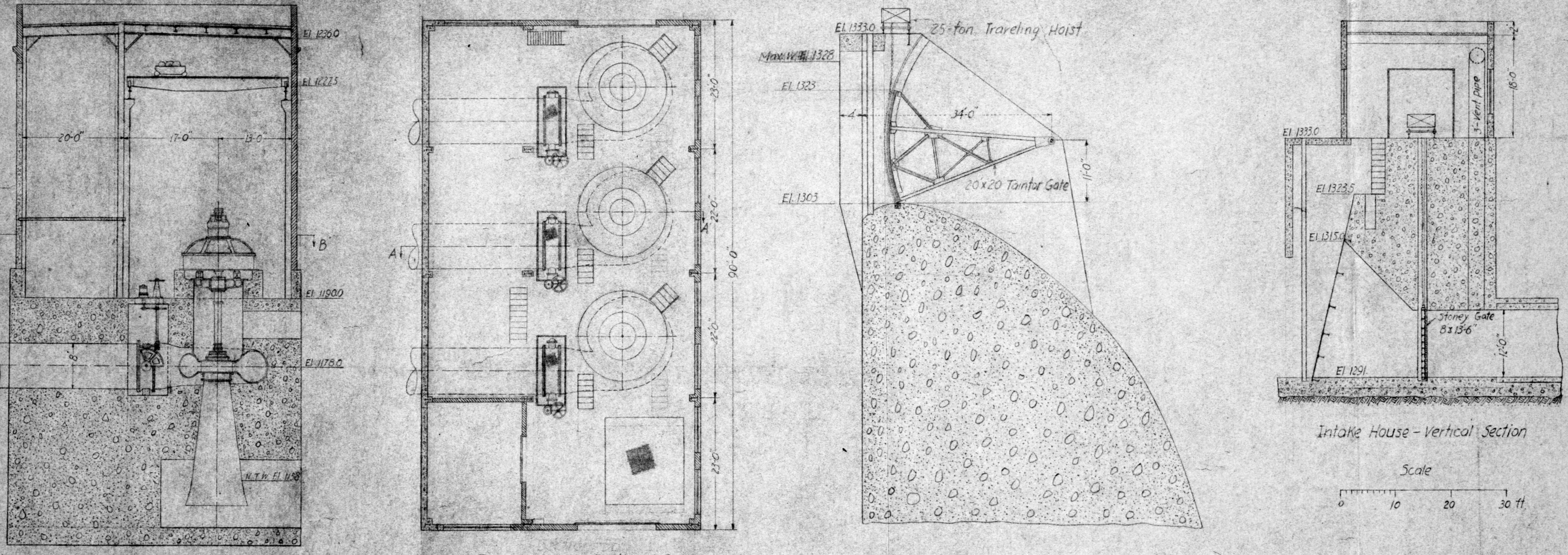
Fig. 15

General Scheme – Plant No. 2, Yung Ting Ho, China



POWER HOUSE, INTAKE AND CREST CONTROLS — PLANT No. 2

Fig 16



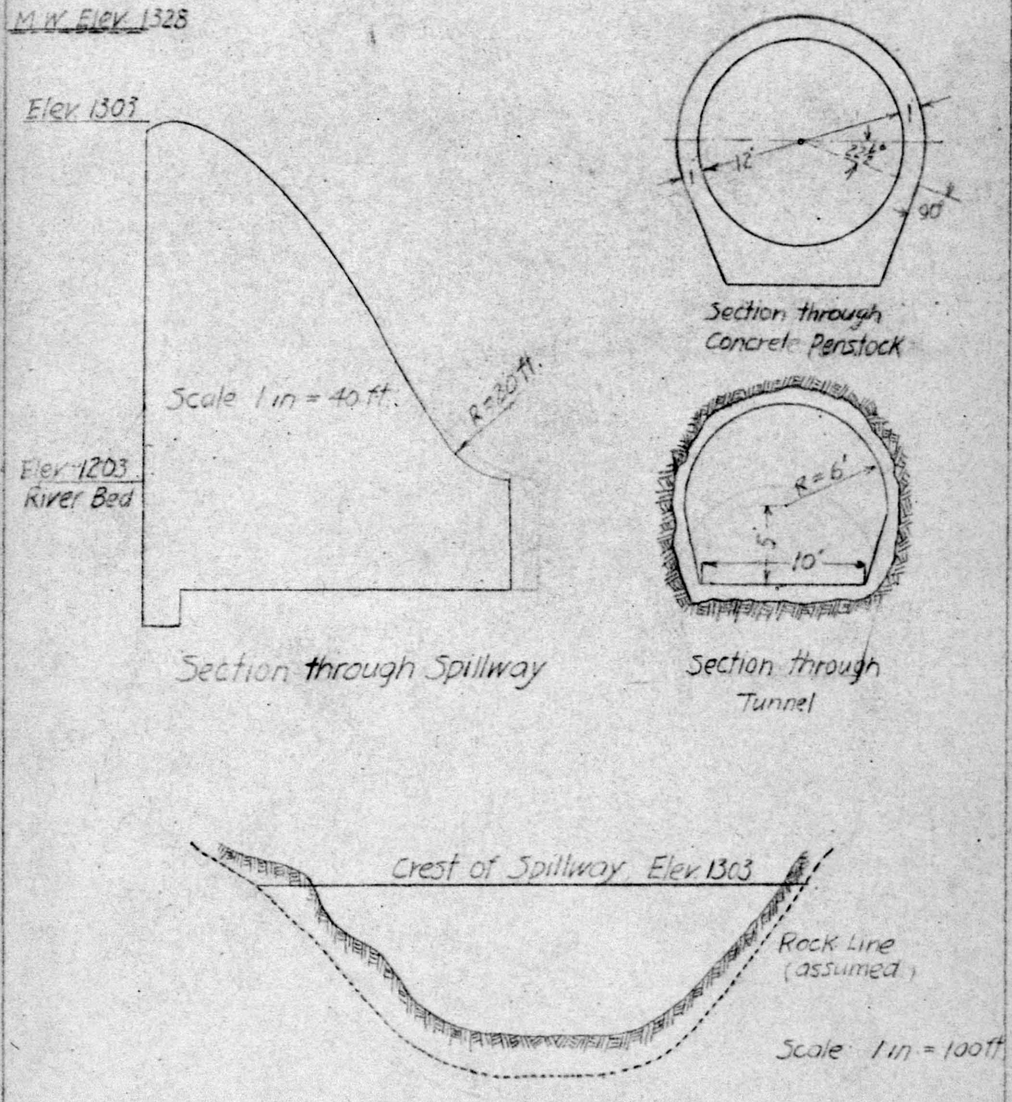
Section AA

Power House - Section BB

Section through Spillway

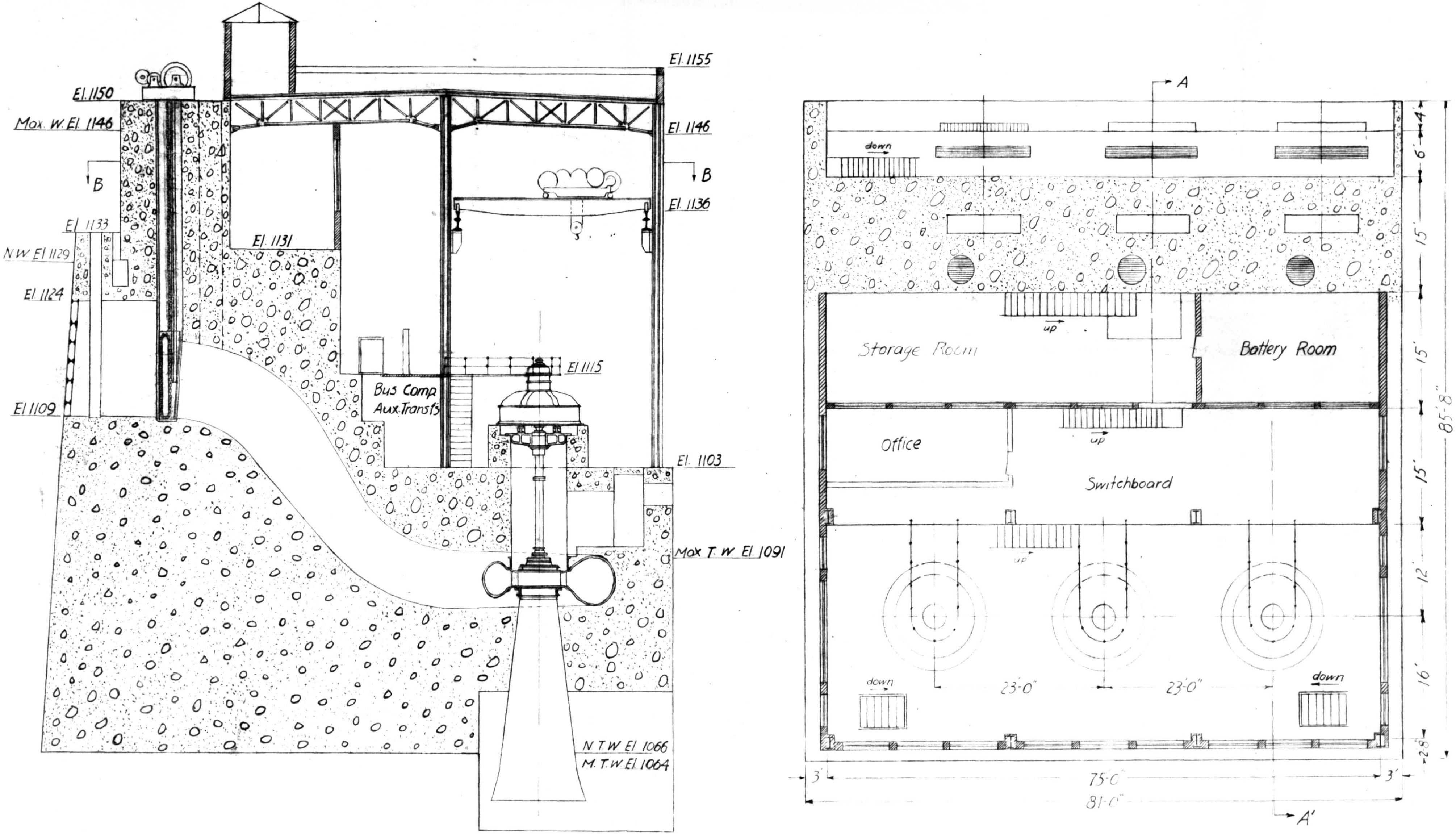
Fig. 17

Dam and Waterway
Plant No 2.



POWER HOUSE — PLANT No. 3. YUNG TING HO, CHINA

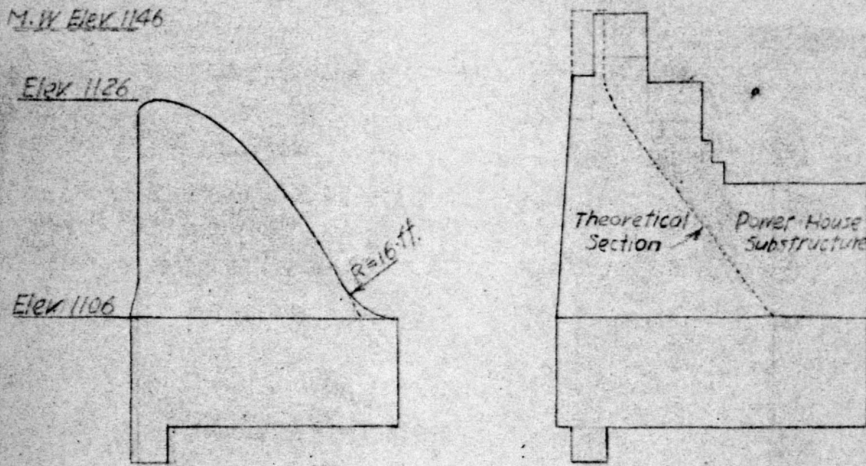
Fig. 18



Section AA'

Section BB'

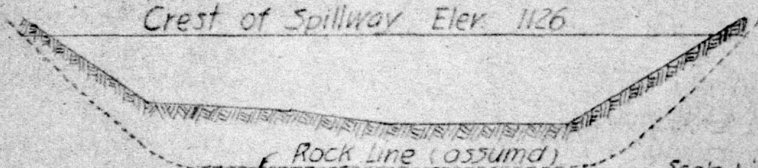
Dam and Dam Site
Plant No. 3



Section through Spillway

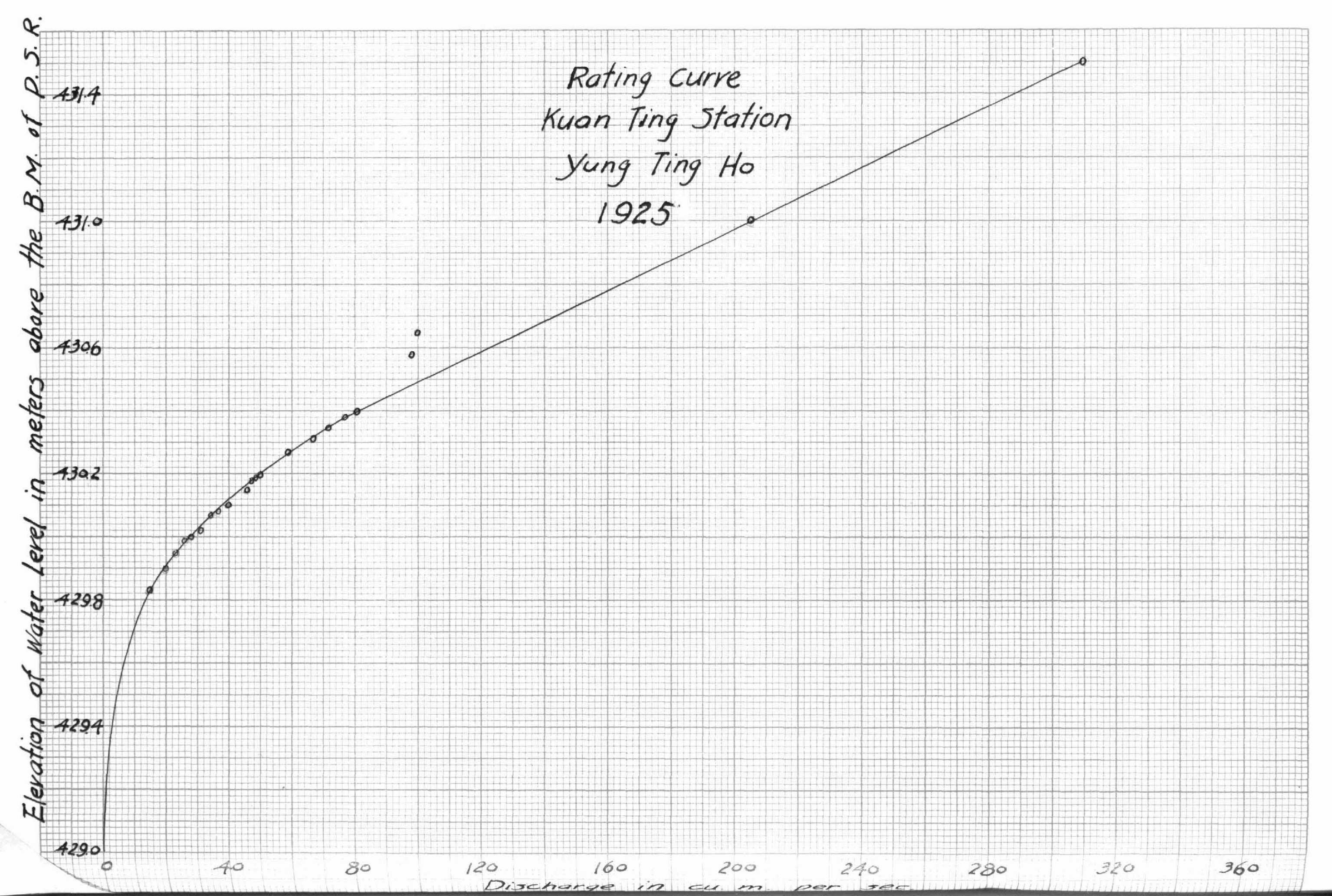
Abutement Section

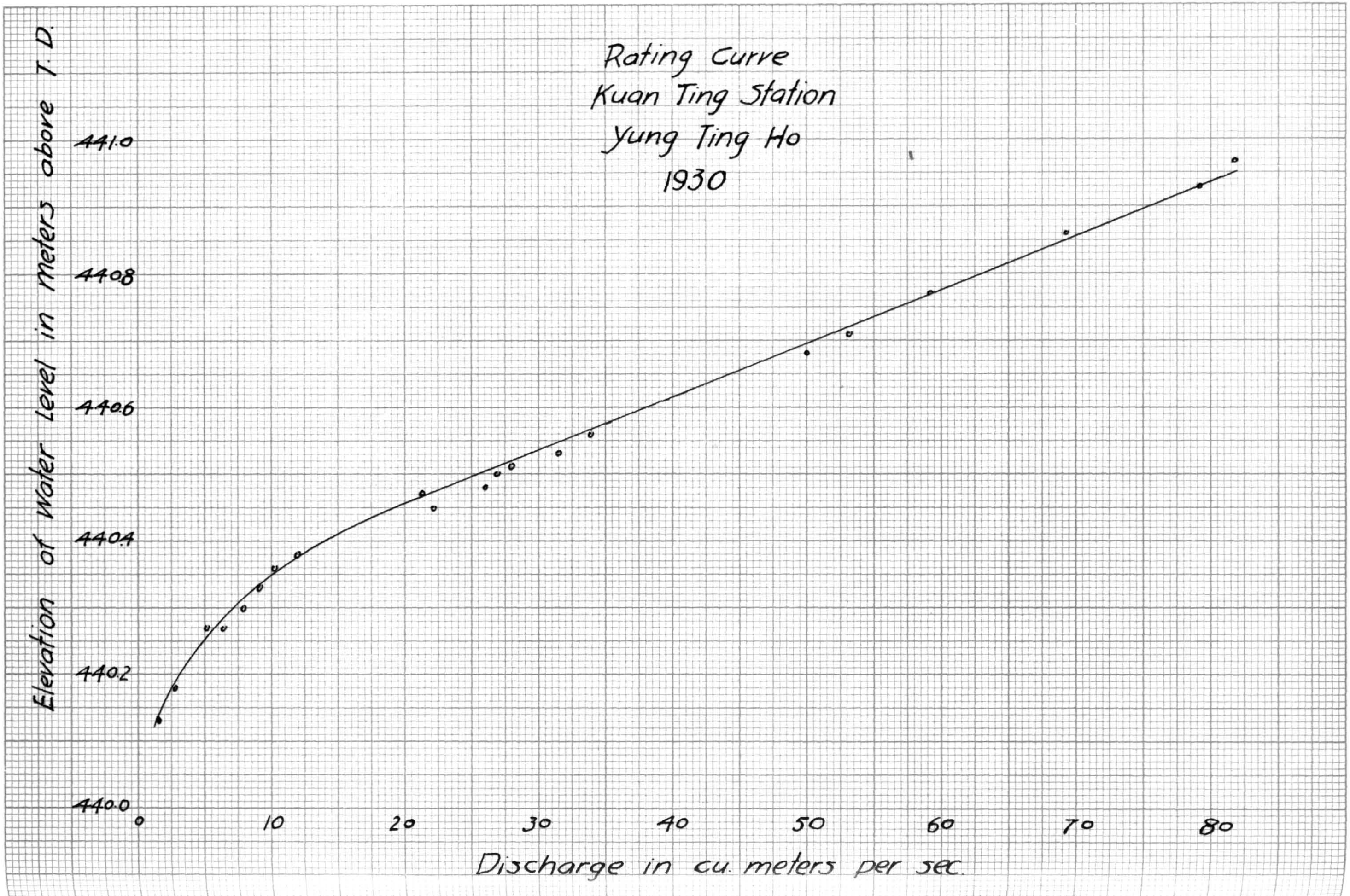
Scale 1 in = 40 ft



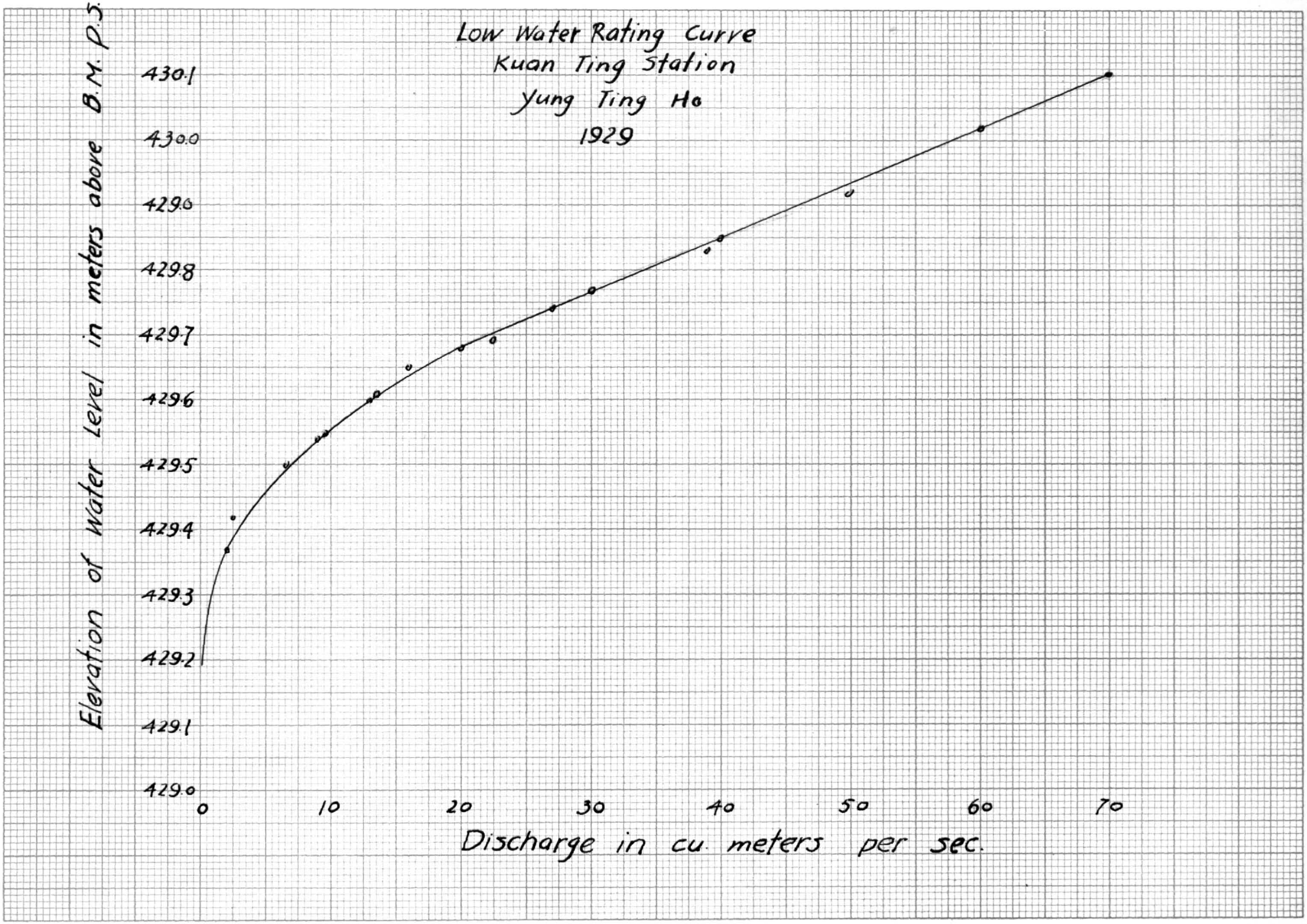
Scale 1 in = 10 ft.

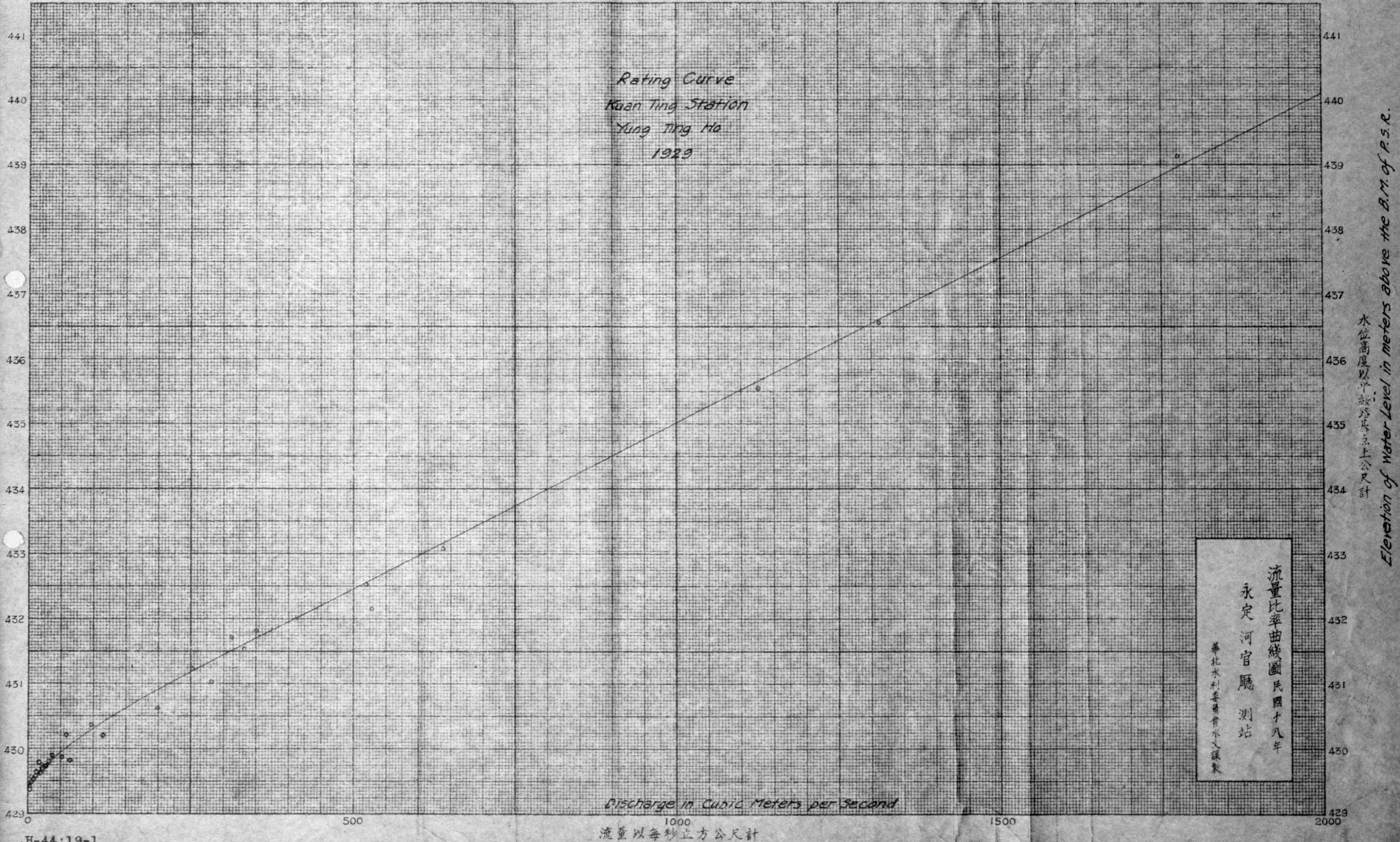
Cross-Section of River — Dam Site


Rating Curve
Kuan Ting Station
Yung Ting Ho
1925
Elevation of Water Level in meters above the B.M. of P.S.R.
431.4
431.0
430.6
430.2
429.8
429.4
429.0
0
40
80
120
160
200
240
280
320
360
Discharge in cu. m. per sec.



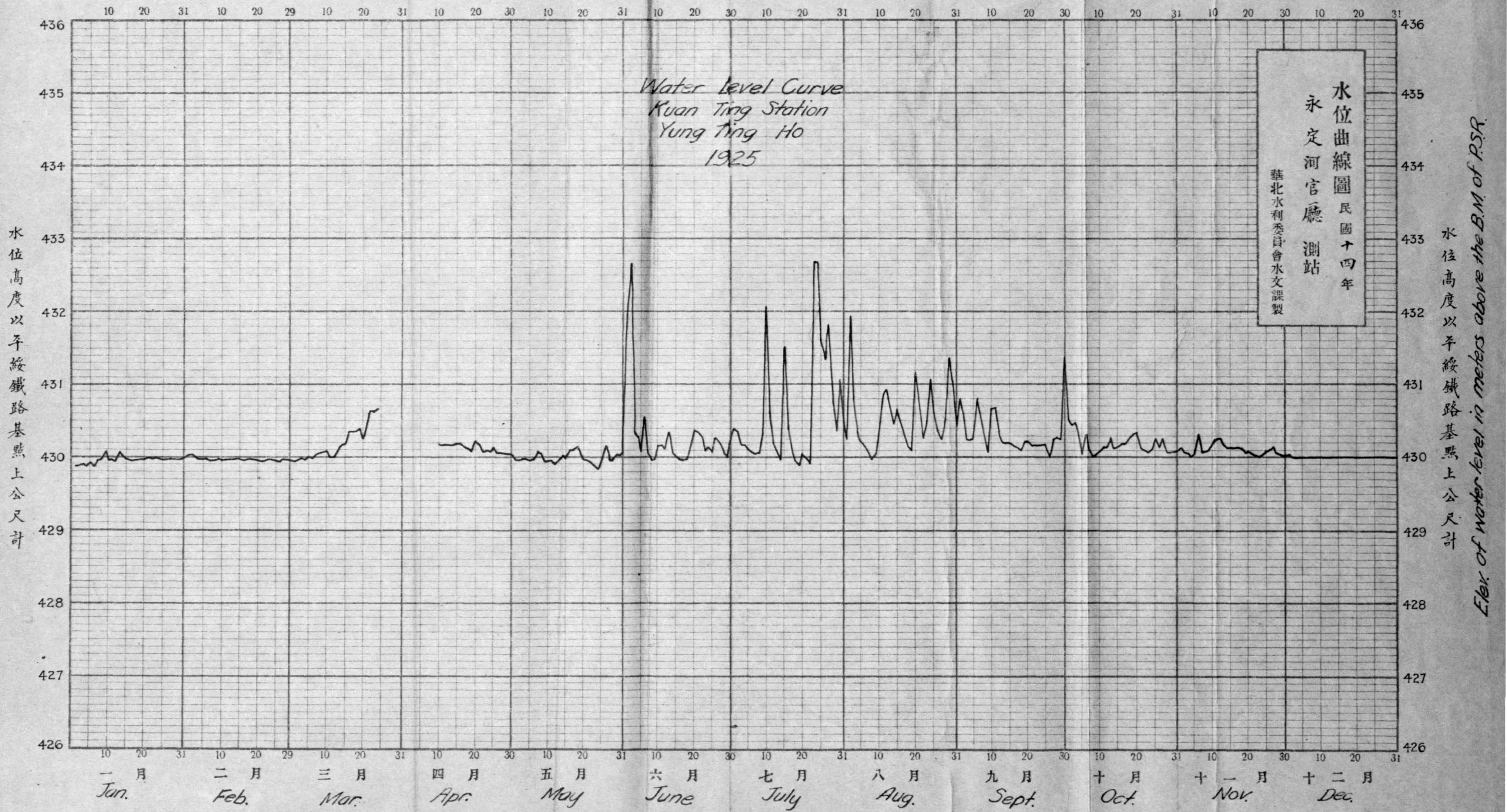
Rating Curve
Kuan Ting Station
Yung Ting Ho
1930
Elevation of Water Level in meters above T. D.
441.0
440.8
440.6
440.4
440.2
440.0
0
10
20
30
40
50
60
70
80
Discharge in cu. meters per sec.


Low Water Rating Curve
Kuan Ting Station
Yung Ting Ho
1929
Elevation of Water Level in meters above B.M. P.S.R.
430.1
430.0
429.9
429.8
429.7
429.6
429.5
429.4
429.3
429.2
429.1
429.0
0
10
20
30
40
50
60
70
Discharge in cu. meters per sec.

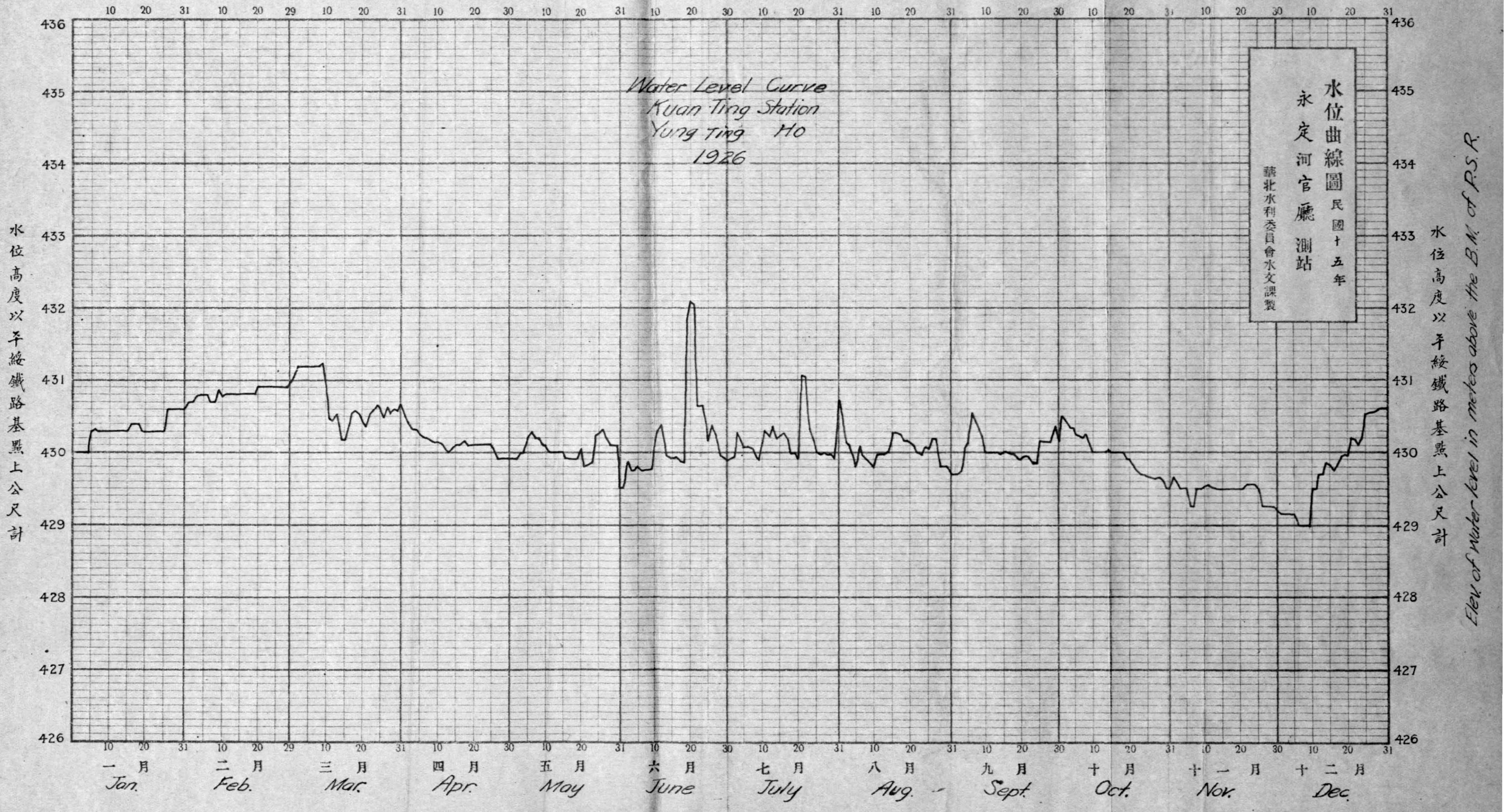


Rating Curve
Kuan Ting Station
Yung Ting Ho
1929
Discharge in Cubic Meters per Second
流量以每秒立方公尺計
Elevation of Water Level in meters above the B.M. of P.S.R.
水位高度以平綏路水准点上公尺計
流量比率曲線圖 民國十八年
永定河官廳測站
華北水利委員會水文課製
0
500
1000
1500
2000
429
430
431
432
433
434
435
436
437
438
439
440
441

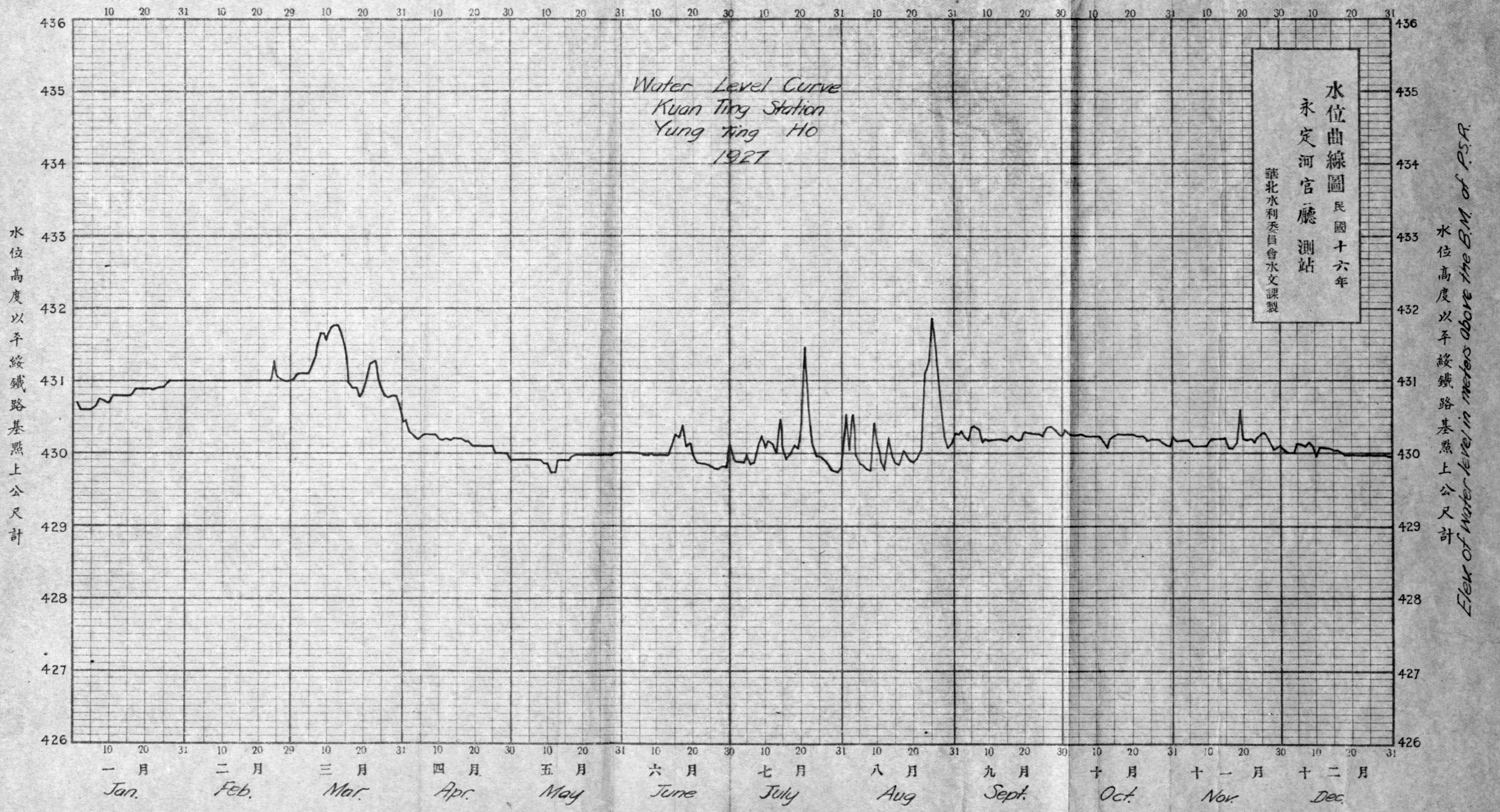
H-44:19-1

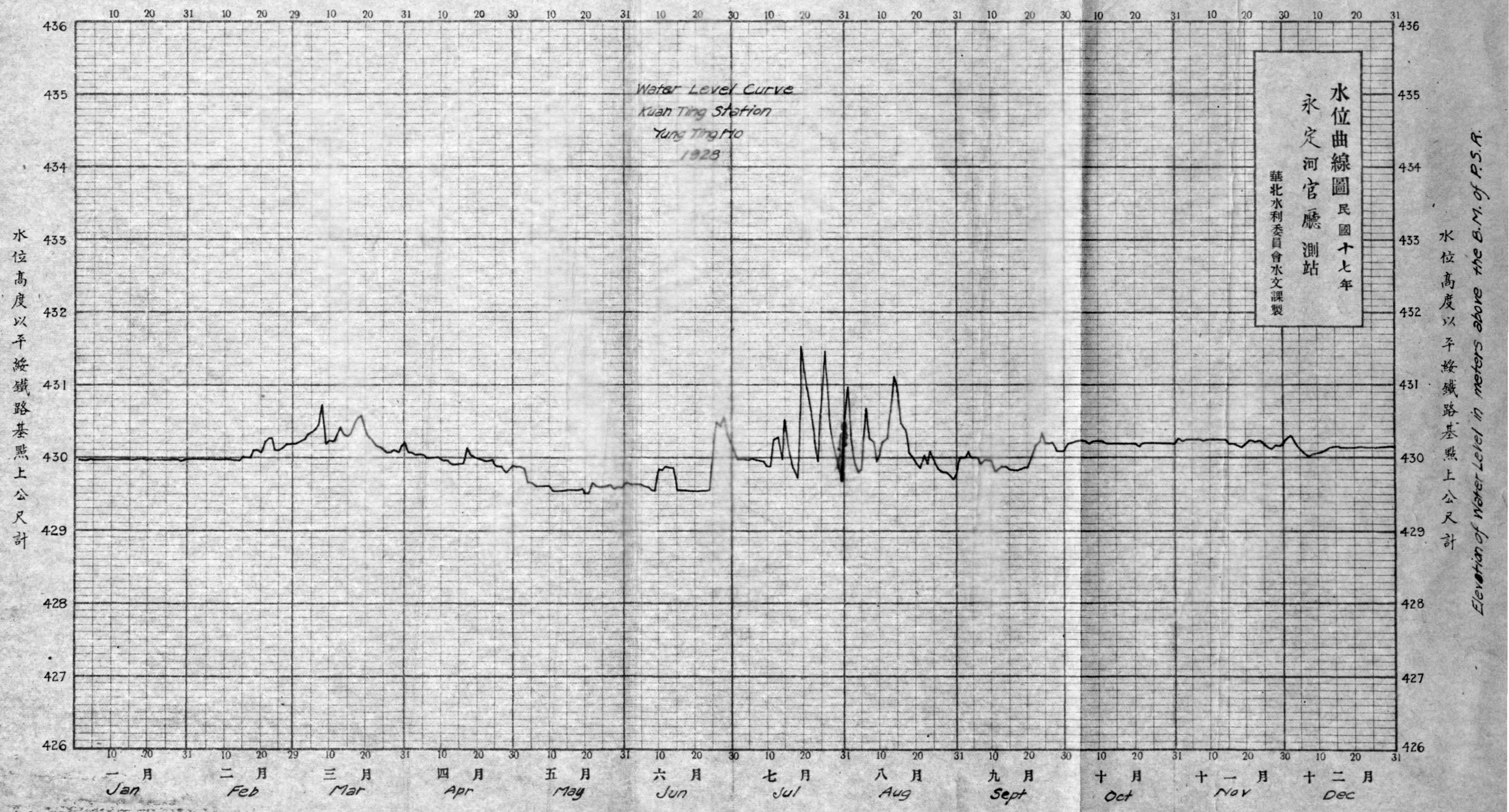


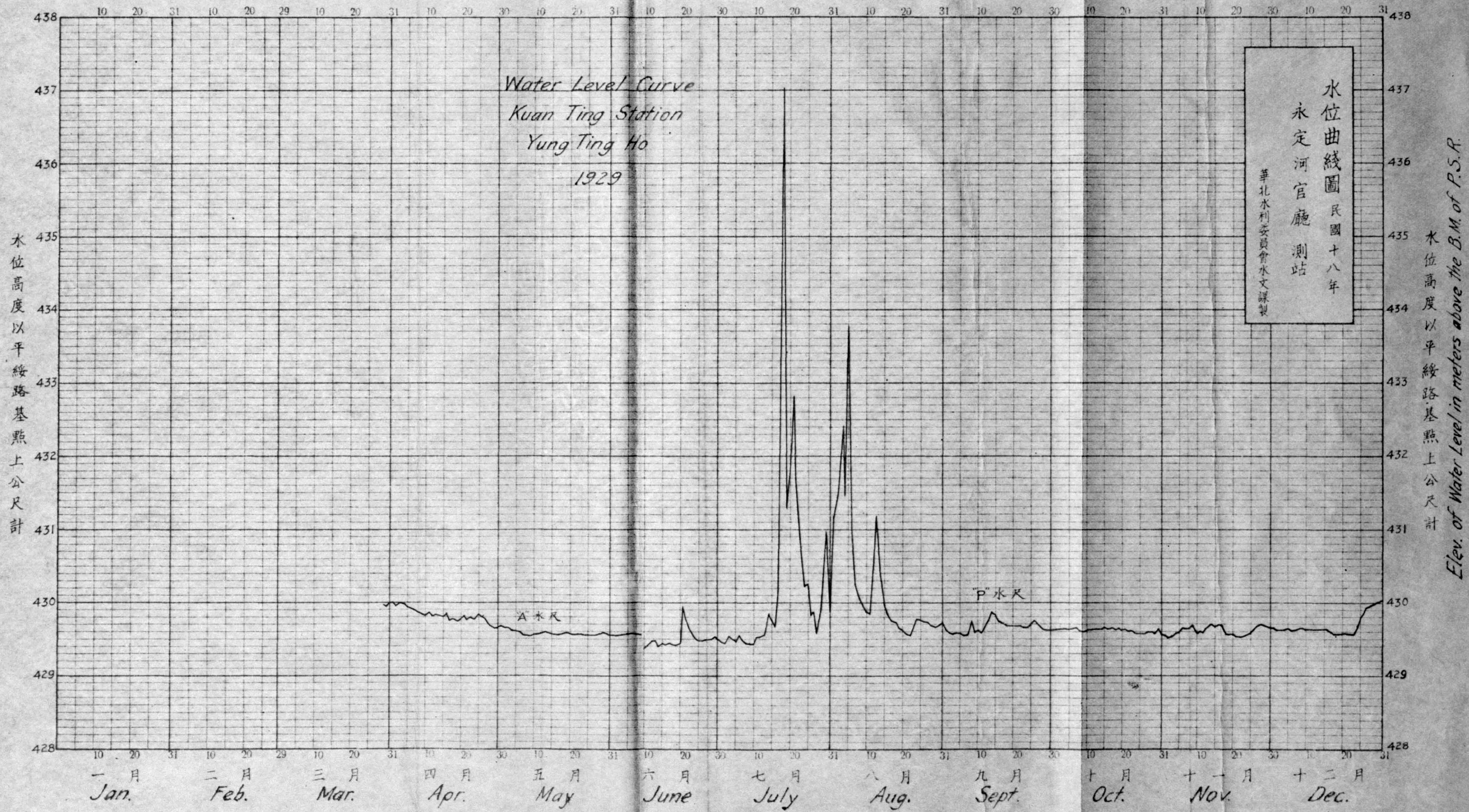
Water level Curve
Kuan Ting Station
Yung Ting Ho
1925
水位曲線圖 民國十四年
永定河官廳測站
華北水利委員會水文課製
水位高度以平綏鐵路基點上公尺計
Elev. of water level in meters above the B.M. of P.S.R.
436
435
434
433
432
431
430
429
428
427
426
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.



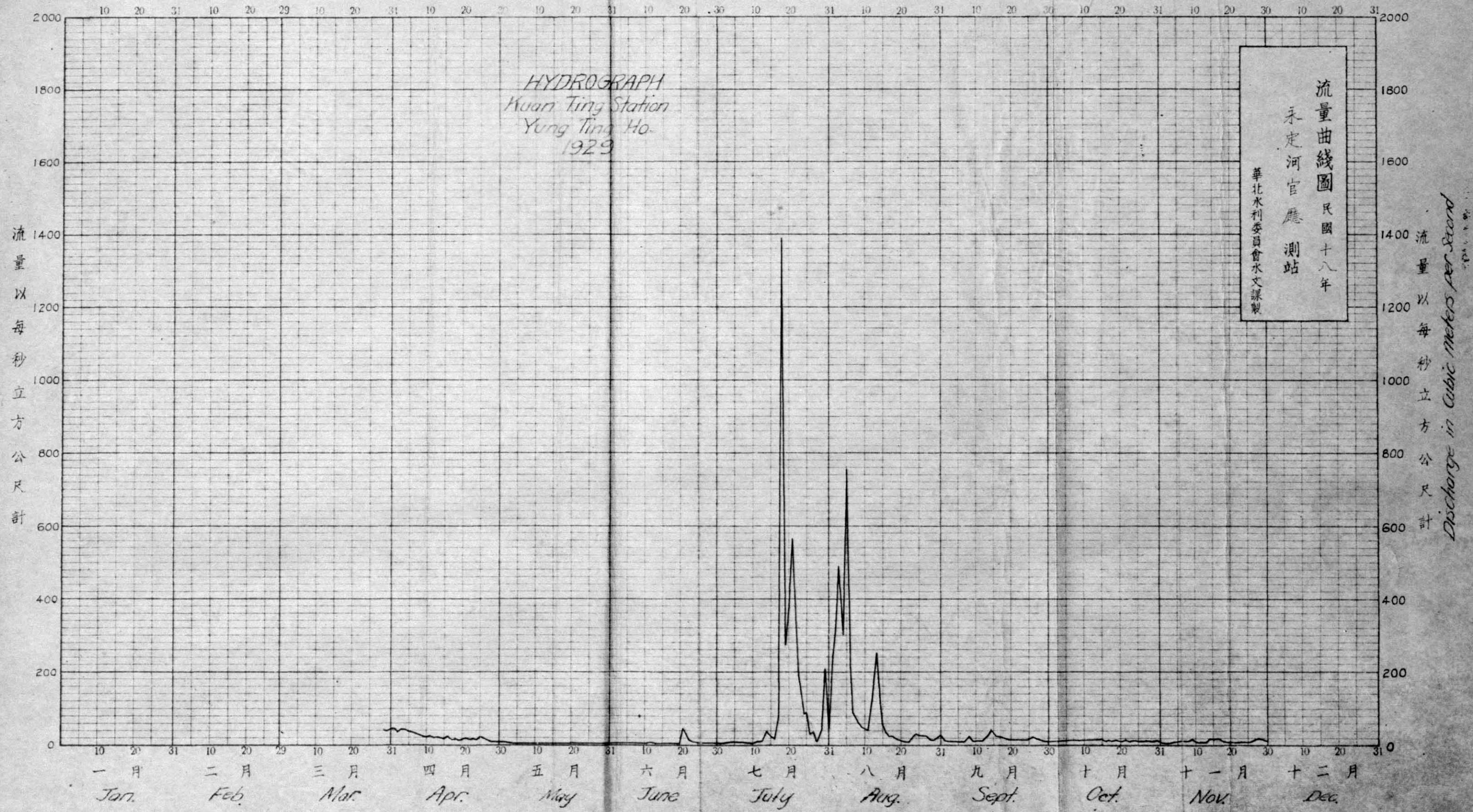
Water Level Curve
Kuan Ting Station
Yung Ting Ho
1926
水位曲線圖
民國十五年
永定河官廳測站
華北水利委員會水文課製
水位高度以平綏鐵路基點上公尺計
Elev. of water level in meters above the B.M. of P.S.R.
436
435
434
433
432
431
430
429
428
427
426
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.


Water Level Curve
Kuan Ting Station
Yung Ting Ho
1927
水位曲線圖 民國十六年
永定河官廳測站
華北水利委員會水文課製
水位高度以平綫鐵路基點上公尺計
Elev. of Water level in meters above the B.M. of P.S.R.
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.
436
435
434
433
432
431
430
429
428
427
426

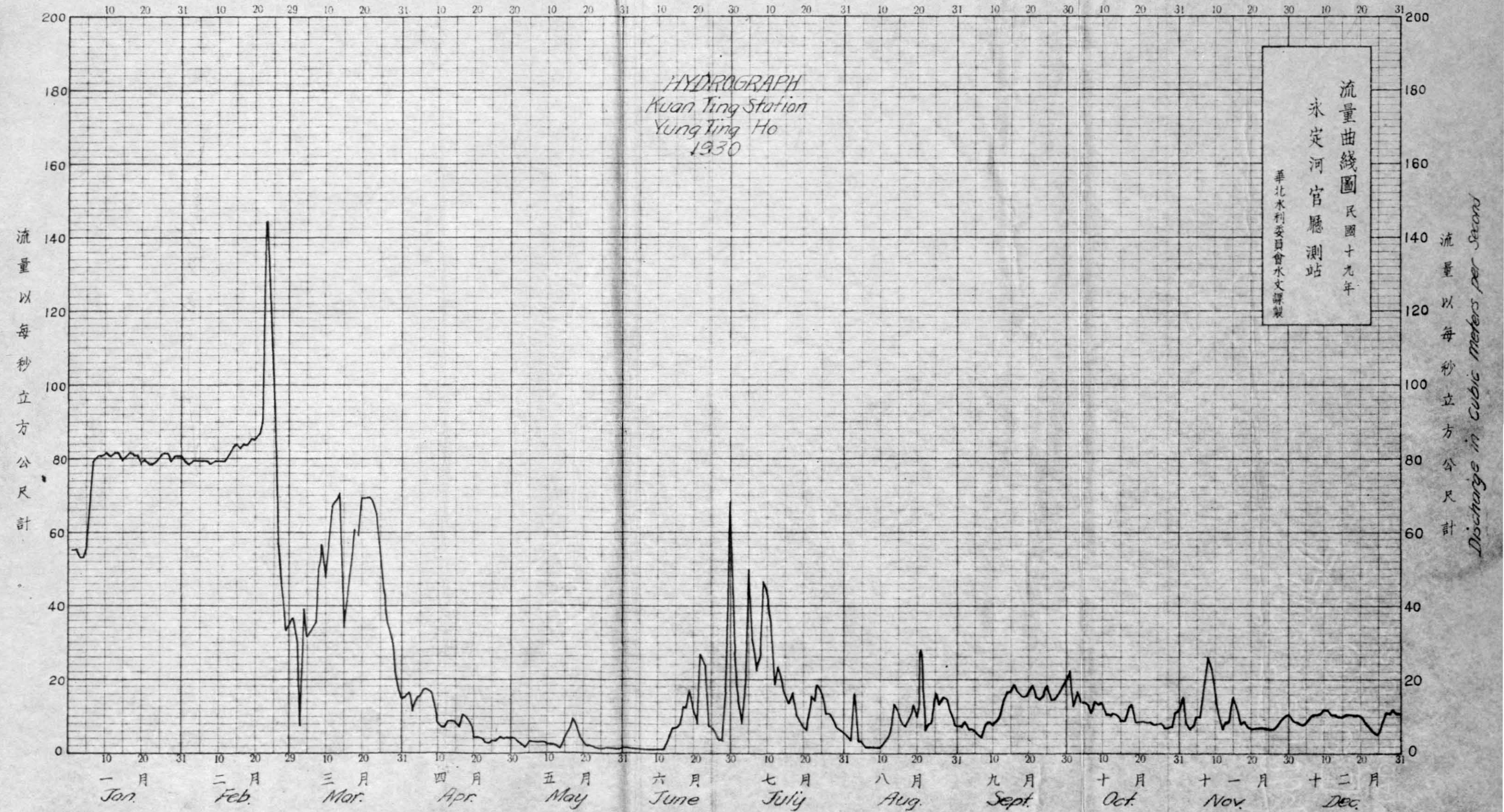

Water Level Curve
Kuan Ting Station
Yung Ting Ho
1928
水位曲線圖 民國十七年
永定河官廳測站
華北水利委員會水文課製
水位高度以平綏鐵路基點上公尺計
Elevation of Water Level in meters above the B.M. of P.S.R.
436
435
434
433
432
431
430
429
428
427
426
一 月 Jan
二 月 Feb
三 月 Mar
四 月 Apr
五 月 May
六 月 Jun
七 月 Jul
八 月 Aug
九 月 Sept
十 月 Oct
十 一 月 Nov
十 二 月 Dec


Water Level Curve
Kuan Ting Station
Yung Ting Ho
1929
水位曲綫圖 民國十八年
永定河官廳測站
華北水利委員會水文課製
水位高度以平綏路基點上公尺計
Elev. of Water Level in meters above the B.M. of P.S.R.
"A"水尺
"P"水尺
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.
438
437
436
435
434
433
432
431
430
429
428

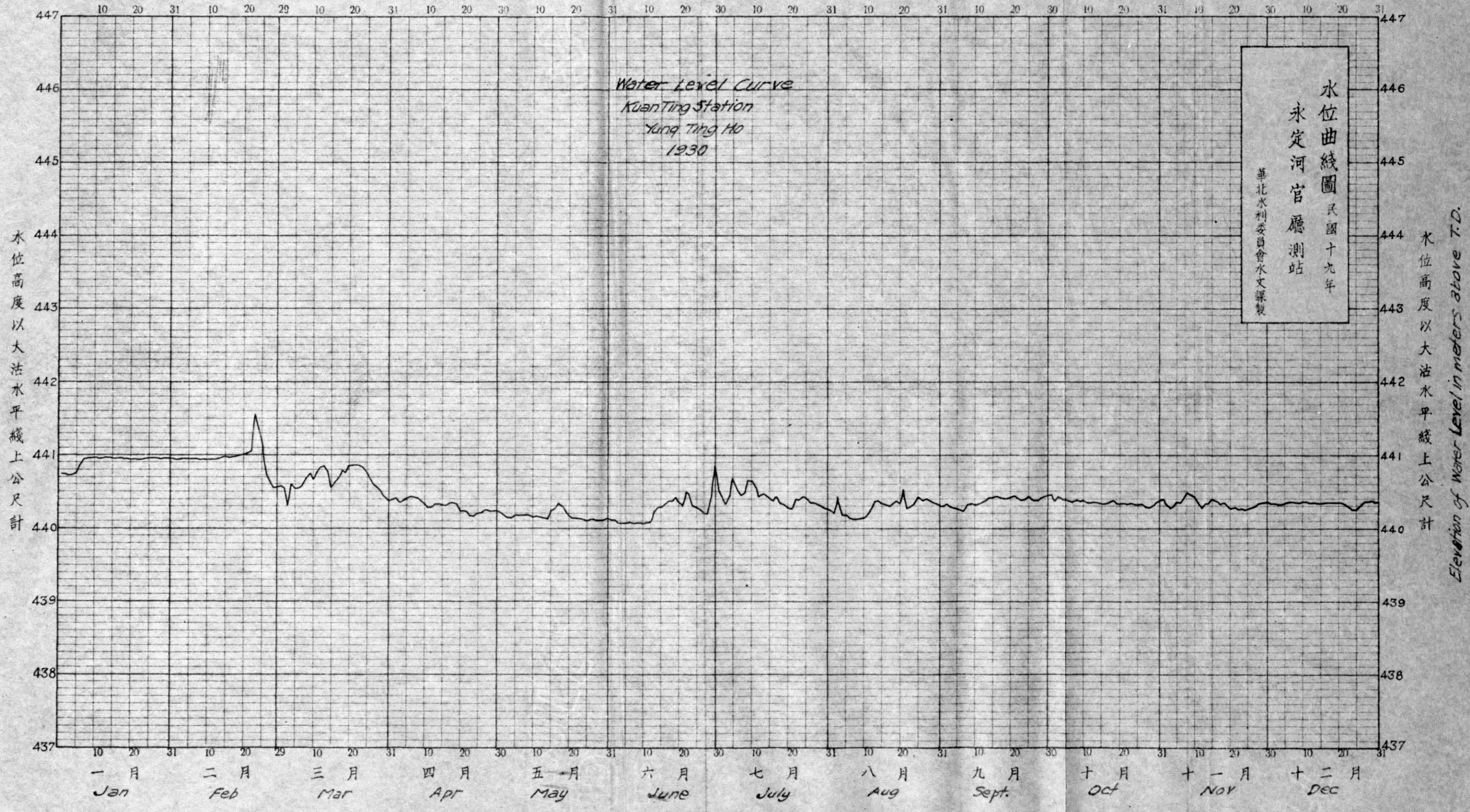
H-5:19-1


HYDROGRAPH
Kuan Ting Station
Yung Ting Ho
1929
流量曲線圖 民國十八年
永定河官廳 測站
華北水利委員會水文課製
流量以每秒立方公尺計
Discharge in Cubic meters per Second
2000
1800
1600
1400
1200
1000
800
600
400
200
0
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.

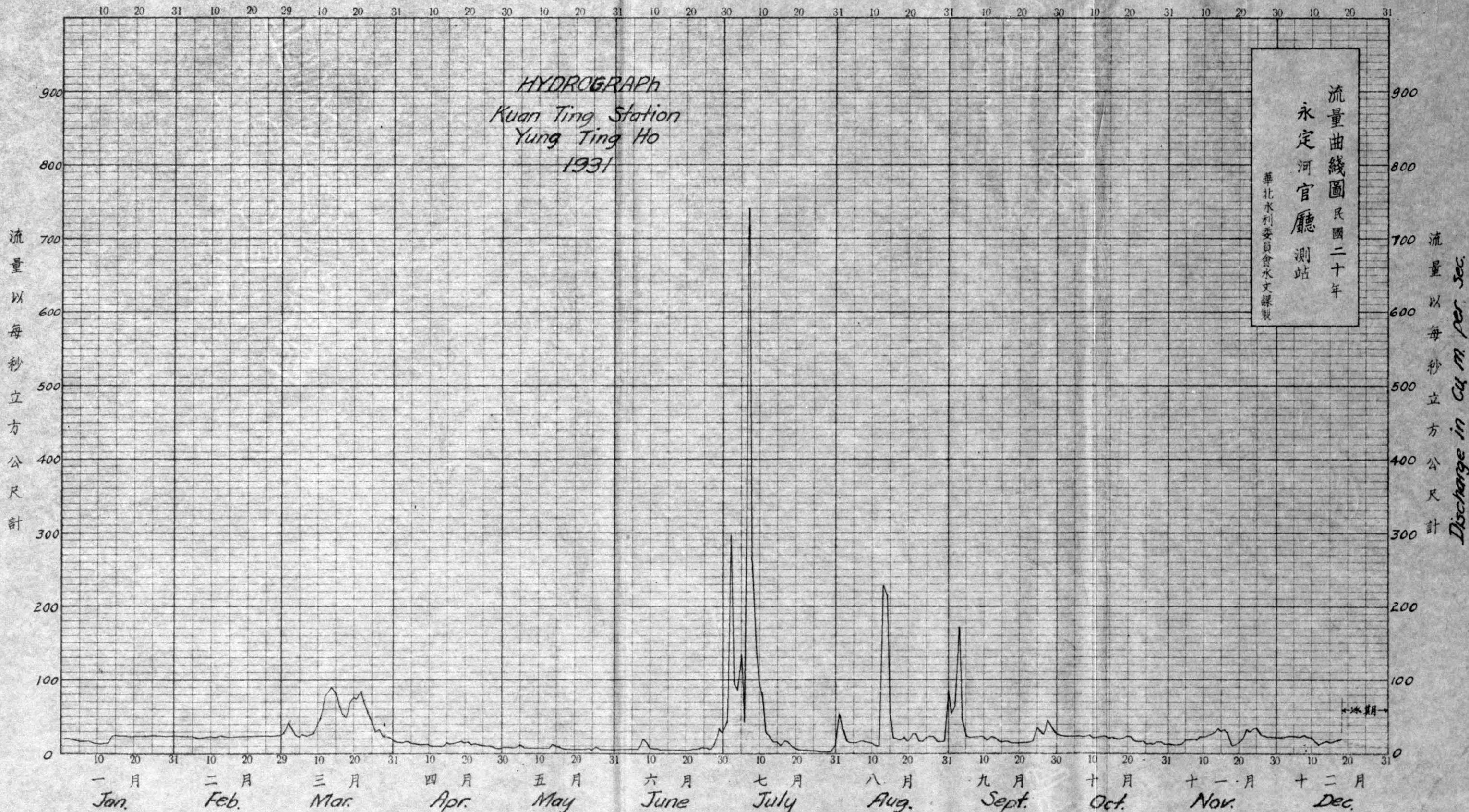
H-9:19-1


HYDROGRAPH
Kuan Ting Station
Yung Ting Ho
1930
流量曲綫圖 民國十九年
永定河官廳測站
華北水利委員會水文課製
流量以每秒立方公尺計
Discharge in Cubic Meters per Second
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.

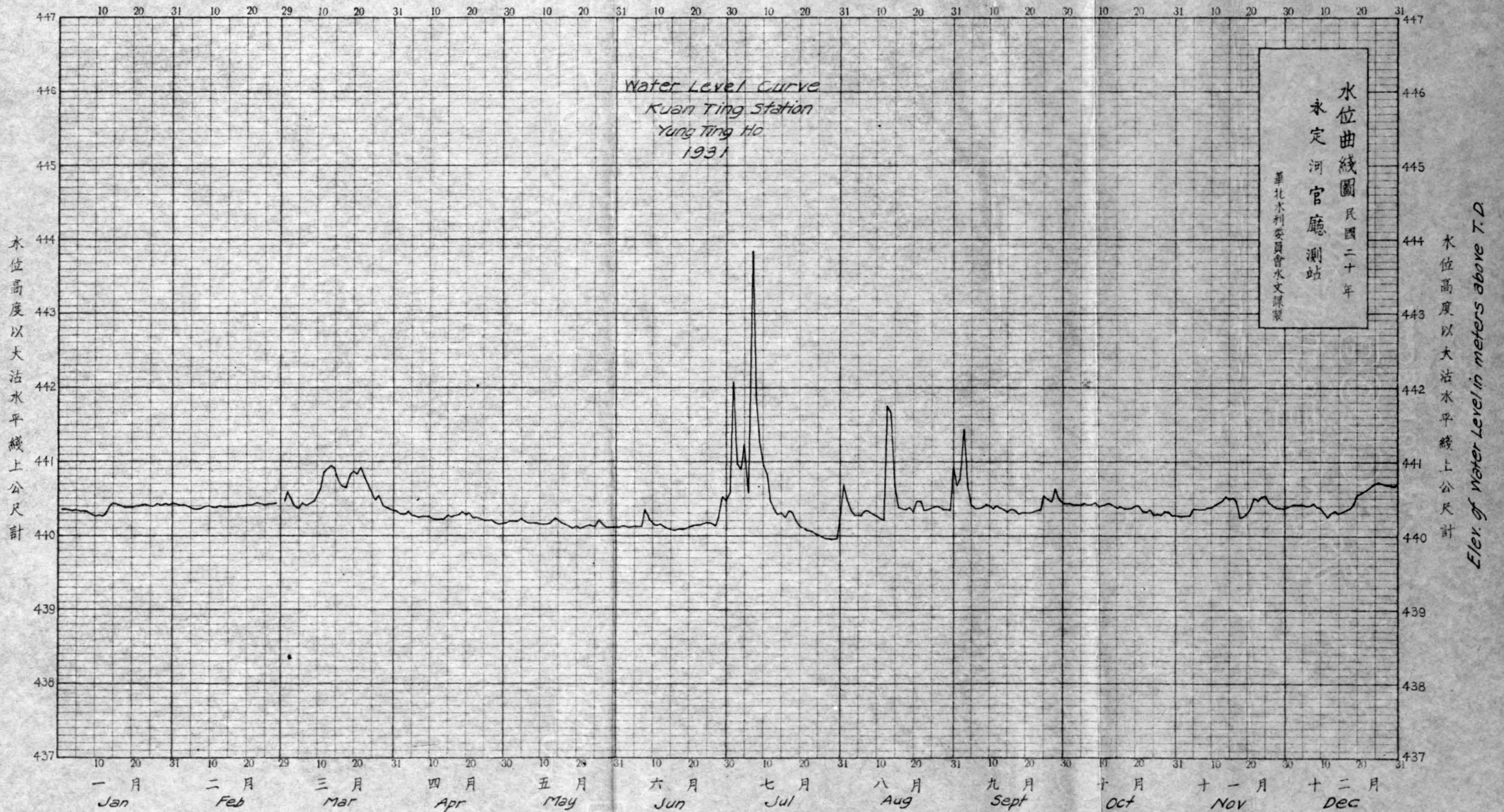
H-9:19-1


Water Level Curve
Kuan Ting Station
Yung Ting Ho
1930
水位曲線圖
永定河官廳測站
民國十九年
華北水利委員會水文課製
水位高度以大沽水平綫上公尺計
Elevation of Water Level in meters above T.D.
一月 Jan
二月 Feb
三月 Mar
四月 Apr
五月 May
六月 June
七月 July
八月 Aug
九月 Sept.
十月 Oct
十一月 Nov
十二月 Dec
447
446
445
444
443
442
441
440
439
438
437

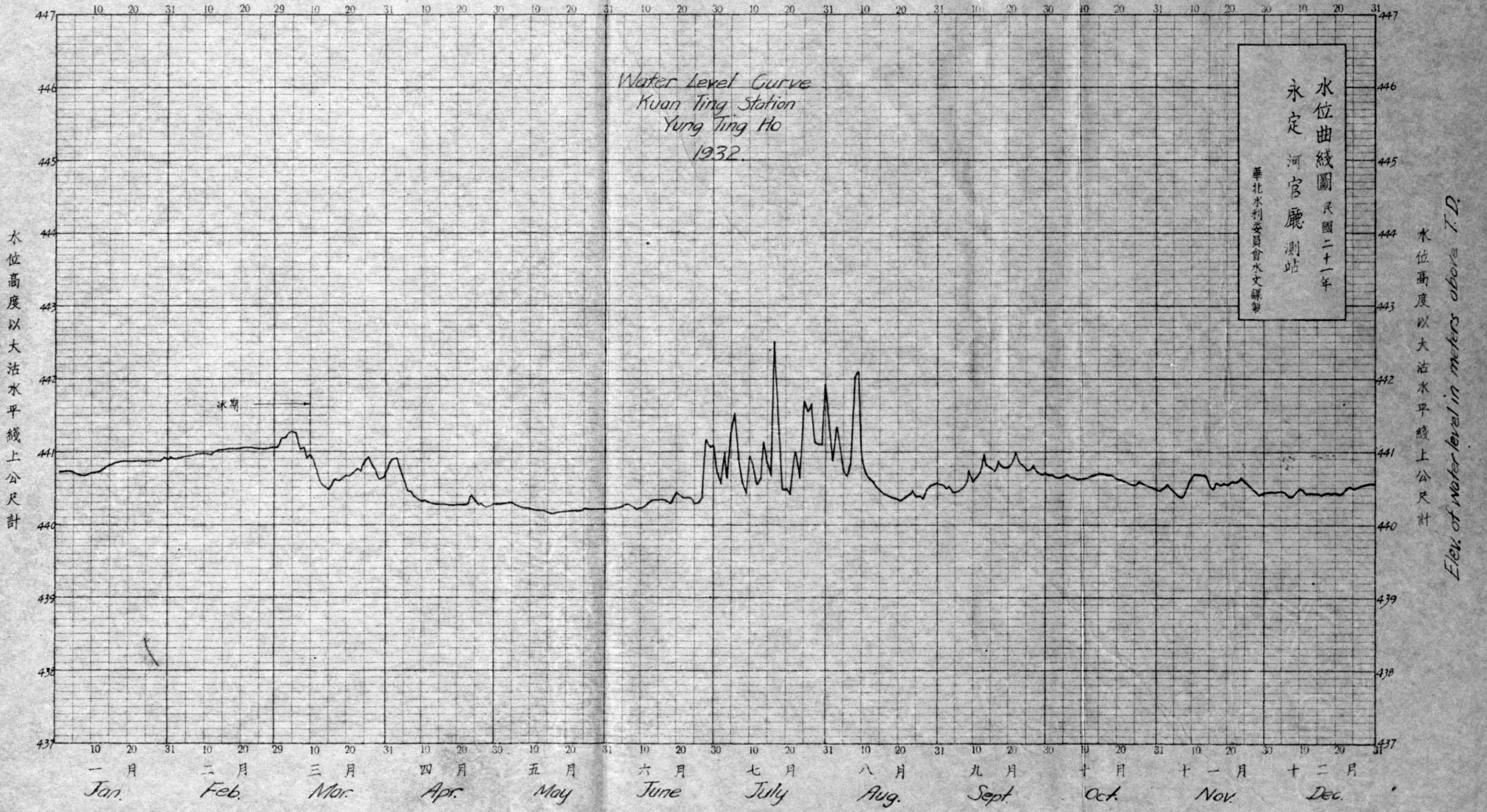
H-5:19-1


HYDROGRAPh
Kuan Ting Station
Yung Ting Ho
1931
流量曲線圖 民國二十年
永定河官廳測站
華北水利委員會水文課製
流量以每秒立方公尺計
Discharge in Cu. M. per Sec.
冰期
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.

H-9:19-1

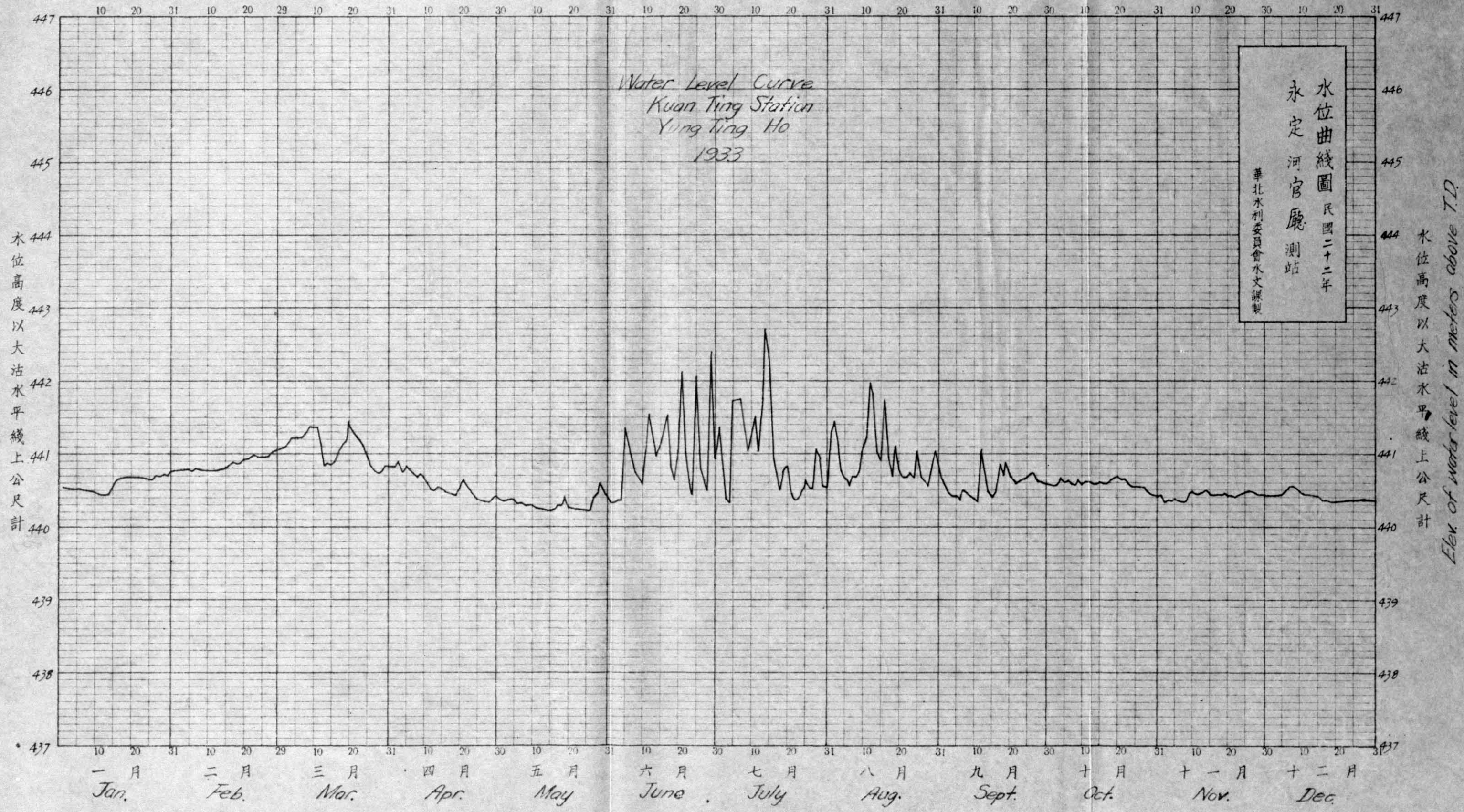

Water Level Curve
Kuan Ting Station
Yung Ting Ho
1931
水位曲綫圖 民國二十年
永定河官廳測站
華北水利委員會水文課製
水位高度以大沽水平綫上公尺計
Elev. of Water Level in meters above T.D.
447
446
445
444
443
442
441
440
439
438
437
一月 Jan
二月 Feb
三月 Mar
四月 Apr
五月 May
六月 Jun
七月 Jul
八月 Aug
九月 Sept
十月 Oct
十一月 Nov
十二月 Dec

H-5:19-1



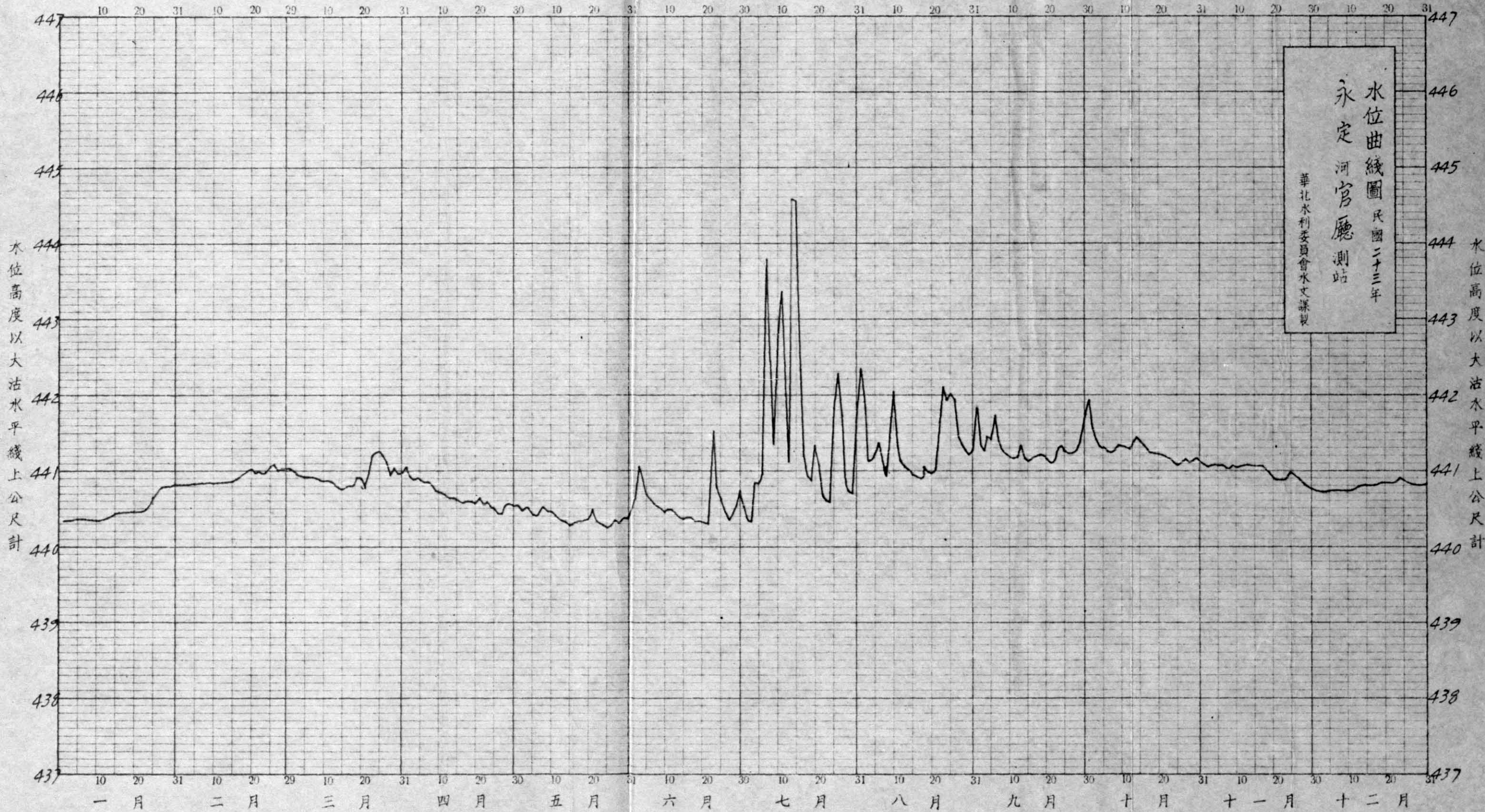
Water Level Curve
Kuan Ting Station
Yung Ting Ho
1932.
水位曲線圖 民國二十一年
永定河官廳測站
華北水利委員會水文課製
冰期
水位高度以大沽水平綫上公尺計
Elev. of water level in meters above T.D.
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.

H-5:19-1



Water Level Curve
Kuan Ting Station
Yung Ting Ho
1933
水位曲線圖 民國二十二年
永定河官廳測站
華北水利委員會水文課製
水位高度以大沽水平綫上公尺計
Elev. of water level in meters above T.D.
447
446
445
444
443
442
441
440
439
438
437
一月 Jan.
二月 Feb.
三月 Mar.
四月 Apr.
五月 May
六月 June
七月 July
八月 Aug.
九月 Sept.
十月 Oct.
十一月 Nov.
十二月 Dec.

H-5:19-1


水位曲綫圖 民國二十三年
永定河官廳測站
華北水利委員會水文課製
水位高度以大沽水平綫上公尺計
447
446
445
444
443
442
441
440
439
438
437
一月
二月
三月
四月
五月
六月
七月
八月
九月
十月
十一月
十二月

H-5:19-1